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GEORGIA INSTITUTE OF TECHNOLOGY
OFFICE OF RESEARCH ADMINISTRATION

RESEARCH PROJECT INITIATION

Date: November 15, 1974

Project Title: Traffice Engineering Services for Fulton County

Project No: E-20-660

Principal Investigator: P. S. Parsonson

Sponsor: Fulton County; Atlanta, Georgia

Agreement Period: From 1/1/74 Until 12/31/74

Type Agreement: Contract dated 7/3/74

Amount: $25,000: E-20-660 $4,030
E-24-629 $8,650
E-24-630 $6,200
E-24-631 $6,120

Reports Required:

Nnne Specified

Sponsor Contact Person (s):

E. T. Hogan
Administrative Assistant
Fulton County Commission
Fulton County Courthouse
Atlanta, Georgia 30303

Assigned to: Civil Engineering

COPIES TO:

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- Dean of the College
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- Other

RA-3 (6-71)

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SPONSORED PROJECT TERMINATION

no

Date: March 13, 1976 OH

Project Title: Traffic Engineering Services for Fulton County

Project No: E-20-660

Project Director: Dr. P. S. Parsonson

Sponsor: Fulton County; Atlanta, Georgia

Effective Termination Date: 12/31/75

Clearance of Accounting Charges: 12/31/75

Grant/Contract Closeout Actions Remaining: None

- ___ Final Invoice and Closing Documents
- ___ Final Fiscal Report
- ___ Final Report of Inventions
- ___ Govt. Property Inventory & Related Certificate
- ___ Classified Material Certificate
- ___ Other ____________________

Assigned to: Civil Engineering (School/Laboratory)

Copies to:

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Division Chief (EES)
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CA -4(3/76)

TRAFFIC ENGINEERING SERVICES
for FULTON COUNTY, GEORGIA

PHASE III

FULTON INDUSTRIAL BOULEVARD

FINAL REPORT

- Inventories
- Analyses
- Recommendations

by

Joseph M. Stupar, Jr.
Graduate Student

Research Projects E20-660 and E20-670

School of Civil Engineering
Georgia Institute of Technology

Project Director: Dr. Peter S. Parsonson,
Associate Professor

AUGUST, 1975

GEORGIA INSTITUTE OF TECHNOLOGY
ATLANTA, GEORGIA 30332

SCHOOL OF
CIVIL ENGINEERING

TELEPHONE:
(404) 894-

September 10, 1975

Mr. David Jennings
Fulton County Traffic Engineer
Public Works Administration, Room 300
165 Central Avenue, S. W.
Atlanta, Georgia 30303

Dear Mr. Jennings:

I am pleased to transmit herewith our Final Report on Fulton Industrial Boulevard. This work completes the third phase of our traffic engineering services rendered to your office over the past several years.

This project, like the previous phases, was carried out primarily by one of my graduate students in partial fulfillment of the requirements for his Master of Civil Engineering degree. The report, by Joseph Stupar, is clearly the work of a student who has performed exhaustive field studies and office analyses with vigor and enthusiasm.

This report considers traffic engineering problems associated with Fulton Industrial Boulevard between the project limits of Gordon Road and Boat Rock Boulevard.

Inventories were conducted of traffic volumes, signal timing, signing and other traffic control devices. Data was also collected relative to speeds and delays, traffic accidents and adjacent land use. Roadway geometrics were inventoried in plan format.

An extensive analysis of the above data included volume/capacity calculations, a review of signal warrants, special displays, signal phasing, detection devices, signal system concepts and access guidelines. Analysis was also performed on accident reports and speed-and-delay information.

Recommendations are centered around providing an overall traffic control plan that will adjust to the short-duration bursts of heavy volumes that characterize industrial land use.

With regard to the I-20 interchange, we have already implemented changes in the signal timing that have eliminated the dangerous back-up of one of the off-ramps onto the freeway. The congestion on the Boulevard during the afternoon rush has been much more difficult to eliminate, but progress has been made through timing changes, improved maintenance of the traffic signal equipment, and improved detectorization. A number of additional improvements in geometrics and traffic control devices are recommended herein for this interchange. Even with these improvements, however, there will continue to be insufficient capacity to handle northbound Boulevard traffic approaching the interchange. This capacity deficiency will become worse with time and could threaten the economic growth of the area. Additional Interstate access needs to be considered through a long-range planning study.

A progressive signal system from Boat Rock Road to Frederick Drive is not considered cost-effective at this time but is recommended for the future. The Gordon Road intersection should continue to function on an isolated basis, with improved phasing and storage to better handle imposed demands. Improvements in signalization, pavement marking and signing are recommended throughout the study area; geometric improvements are recommended where required, both for short- and long-range improvements.

Our observations at Fulton Industrial Boulevard have been similar to those at Roswell Road in Sandy Springs in that the control equipment shows strong evidence of insufficient maintenance. This is a reflection not on the quality of the Fulton County electrical technicians but rather on their quantity. There are not enough traffic signal technicians on your staff. Our letter to your office dated October 1, 1973, recommended that the existing two technicians be used exclusively for traffic-signal controllers and detectors, and that an additional technician be employed to perform the other electrical work (including lamp replacement and repair of signal heads). The more time-consuming bench repairs of controllers should be contracted out to local technicians as required by seasonal work loads associated with lightning damage.

Fulton County has committed itself to the use of relatively complex, traffic-responsive signals, rather than the simpler, fixed-time type. This commitment by the County will be a service to the traveling public only if the equipment operates as it was designed to operate. In the absence of proper lightning protection, preventive maintenance and timely repairs, the equipment will only be a source of danger, delay and angry telephone calls and letters.

Georgia Tech is most grateful for this opportunity to be of some service to Fulton County. Four of our graduate students have received experience that could not be duplicated in the classroom or laboratory. They have all gone on to success as practicing traffic engineers and transportation planners. We are eager to a renewal of our agreement that will extend our cooperation through 1976.

Yours very truly,

Peter S. Parsonson, P. E.
Assoc. Professor

Encl.
cc: Office of Contracts Administration

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INTRODUCTION

The introduction to this report discusses present conditions necessitating a traffic engineering study for Fulton Industrial Boulevard. Project limits, objectives, and the scope of work are also reviewed.

Project Background

Fulton County is responsible for traffic engineering in those parts of the County that are not included within incorporated areas. The County relies on consultants for assistance with its more rigorous problems. In this context, the Georgia Institute of Technology's School of Civil Engineering has been retained on a continuing basis to provide traffic engineering services.

Following a meeting early in February, 1975, with Fulton County staff members, it was determined that the priority project for 1975, should be a portion of the Fulton Industrial Boulevard (S.R. 70). The project limits extend between and include the intersections of the Boulevard with Gordon road and Boat Rock Road. Figure 1 depicts the general study area.

Development along the portion of the Boulevard north of Cascade Road (S.R. 154) is of relatively high density and is composed primarily of service companies and light industry sites. South of Cascade Road, adjacent land is largely undeveloped at present but is expected in the future to become similar to the north portion. The Boulevard is designated as both S.R. 70 and S.R. 154 south of Cascade Road.

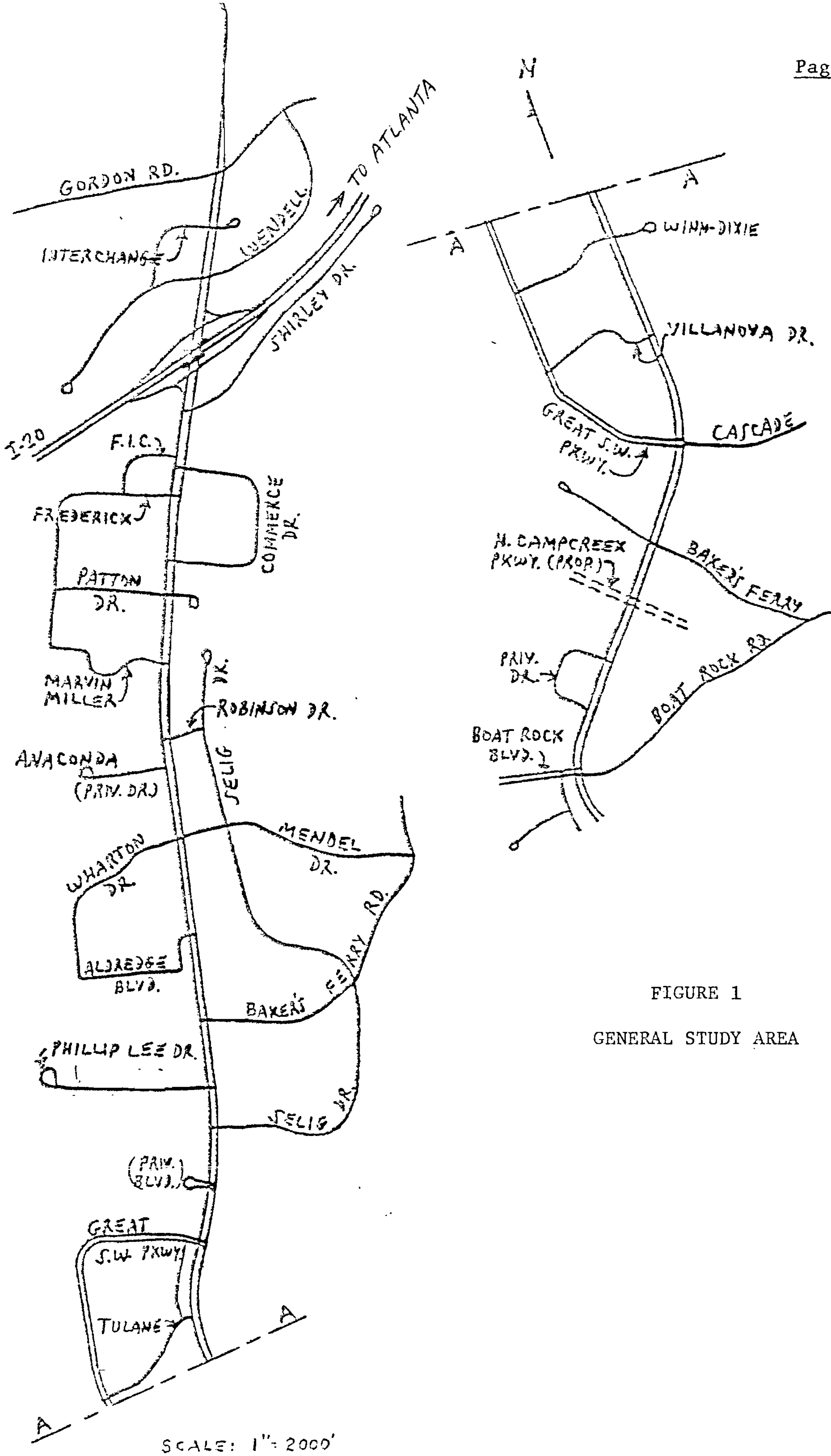


FIGURE 1

GENERAL STUDY AREA

Study Objectives

In a letter dated March 17, 1975 County Manager Sam Brownlee stated that a study was needed to ascertain possible traffic improvements along the Boulevard within the aforementioned study limits. The County Manager noted that the section of greatest concern was that bounded by the signalized ramps of I-20 and the Great Southwest Parkway T intersection. He further indicated that alterations might include "additional signals, intersection improvements, and coordination of signals." The County Manager also mentioned the need to "reduce accidents and delay."

During the meeting of February 5, 1975 with Mr. Jake Ivey and Mr. David Jennings, accident and congestion problems were stressed. Sporadic demands by side streets and the need for coordinated traffic flow were also discussed.

The following paragraphs indicate specific problems facing the consultant in attempting to remedy the Boulevard's traffic situation:

Presently, there are six signalized locations within the study section. These include Gordon Road, the Interstate-20 ramps (two locations), Patton Drive, Wharton Drive, the Great Southwest Parkway (T intersection) and Cascade Road. All existing locations are eventually planned to be full-actuated. The I-20 intersections are already full-actuated and loop detectors are planned to be installed at the remaining locations by the County. Presently, Patton Drive is operating on a semi-actuated basis. These signalized locations need to be assessed to determine the need, if any, for a coordinated signal system controlling all or a portion of the intersections. In addition, the intersections require a determination of their functional efficiency on an isolated basis (ie. capacity studies).

Additional requests are likely to be forthcoming from various properties abutting Fulton Industrial Boulevard to provide traffic control devices (esp. signalization) at existing or approved median openings. This has already been the case at the private road serving Anaconda. Such requests should be evaluated on a sound basis which considers both the need for signalization and the effect it could have on an adjacent, coordinated, system. The proposed North Campcreek Parkway intersection(s) with the Boulevard will eventually require similar consideration.

A number of complaints have been received by Fulton County staff concerning access problems to driveways where no median opening currently exists. Certain facilities whose drives open on the Boulevard can only be reached via "U" turns at the nearest adjacent median cut. Recent studies dealing with driveway and median access controls may dictate the need to change or establish additional standards.

Specific storage problems presently exist at the I-20 diamond interchange. During the morning peak period the westbound I-20 ramp to the Boulevard experiences congestion. During the evening peak period capacity problems occur for northbound Boulevard traffic bound for both destinations to the east and west via I-20. A fully actuated system is currently operational; fine tuning of the system, as well as geometric improvements, may be required.

Numerous collisions, of the cross-movement and rear end types, occur on a frequent basis. Accident records need to be evaluated to determine causal factors. Corrective measures should then be taken with the intent to significantly reduce accident occurrence.

Study Procedure

An "Activity Flow Chart" has been prepared (see figure 2) which indicates a typical sequencing of events for studies of this type. Citizen input is directed to the client staff (Fulton County). The client then engages the service of a research team to clarify needs and provide problem-solving recommendations. Following an initial meeting, the research group then proceeds to review applicable literature and investigates the study site. Data is collected and base plans are prepared for analysis. A field check should be conducted to insure conformity of base plans to actual conditions. A literature review, applicable to problems thus far formulated, parallels data collection and is also utilized in the problem analysis. Concurrent with analysis is a study of reference material related to design requirements anticipated for solutions. Finally, the recommended report and plans will be submitted to the client for approval and implementation.

A feedback process should occur throughout applicable portions of the activity sequence to maintain liason between the client and research team.

A detailed breakout of each stage in the generalized flow chart is discussed below:

- INITIAL MEETING: At this meeting a site is selected, and the project limits and scope of services are defined.
- INITIAL SITE INVESTIGATION: This would include observation of existing traffic control devices and their effectiveness during peak travel periods. Also noted would be storage and channelization bottlenecks.
- LITERATURE REVIEW: A study of background material will be conducted to ascertain applications and techniques which may be applicable to to the client's project.

GENERALIZED ACTIVITY FLOW CHART

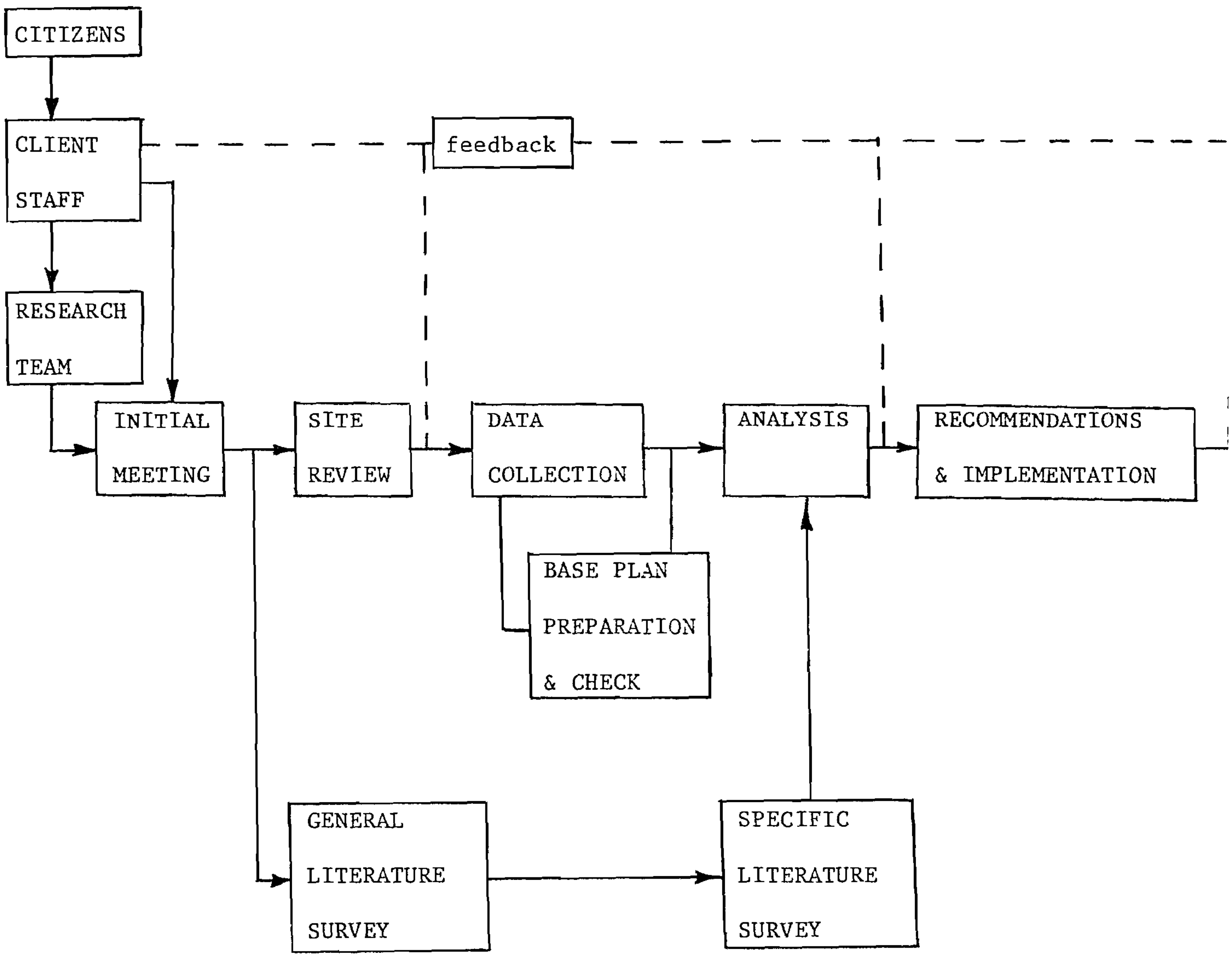


FIGURE 2

ACTIVITY FLOW CHART

- DATA COLLECTION: Existing roadway conditions will be surveyed or assembled relative to geometrics, traffic signal timings and equipment, and peak period traffic counts. Transit stops etc. will be noted. Accident statistics will be reviewed and collision diagrams prepared.
- BASE PLAN PREPARATION: Within the roadway right-of-way, plans will be prepared showing the general study area and specific intersections to be analyzed. Existing traffic control devices, utilities, channelization and applicable pavement markings will be recorded. Adjacent land use will also be included.
- FIELD REVIEW: The base plan should be checked against field conditions to insure conformity. To the extent possible, proposed recommendations should be referenced to determine their feasibility. Any recent field development should be noted for inclusion in the base plan and/or for its effect on the study analysis and recommendations.
- ANALYSIS: A compatible traffic engineering solution will be devised using data based on traffic trends, signal warrants, capacity, safety requirements, and land use. Future development will be considered insofar as possible. Safety aspects to be evaluated could include sight distance, geometrics, gap availability, etc.
- LITERATURE SURVEY: With an understanding of specific problems, determination of equipment, phasing diagrams and traffic control device locations can be determined based on text references and professional assistance.
- RECOMMENDATIONS AND IMPLEMENTATION: A report will be prepared for submission to the client agency which, with enclosed plans, will provide a working solution.

SECTION I

-INVENTORIES-

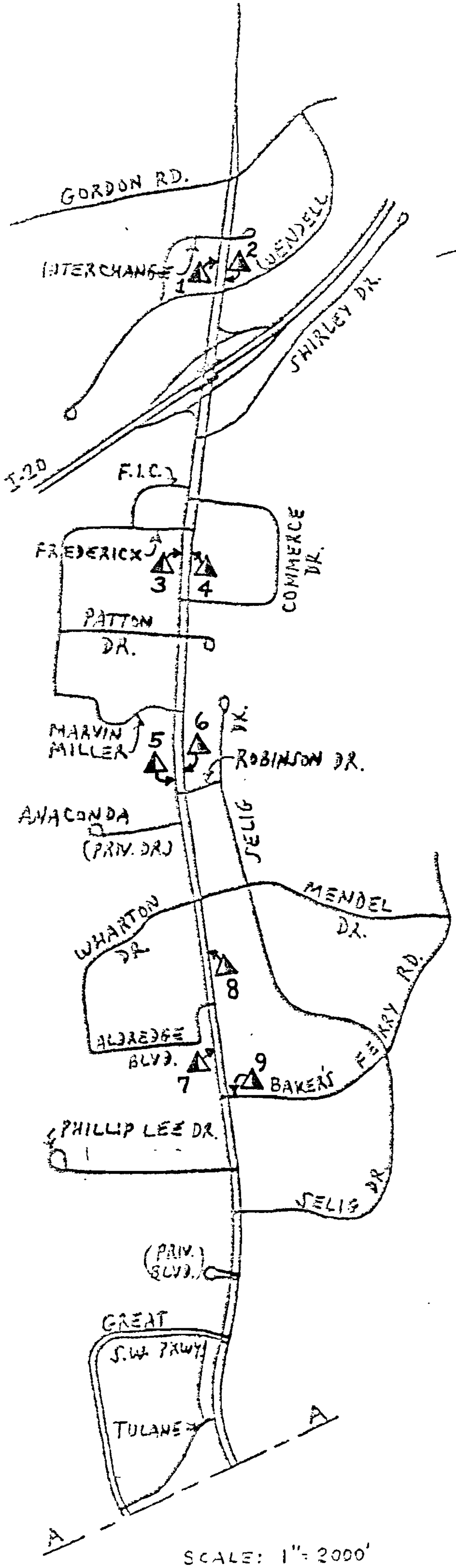
INVENTORIES

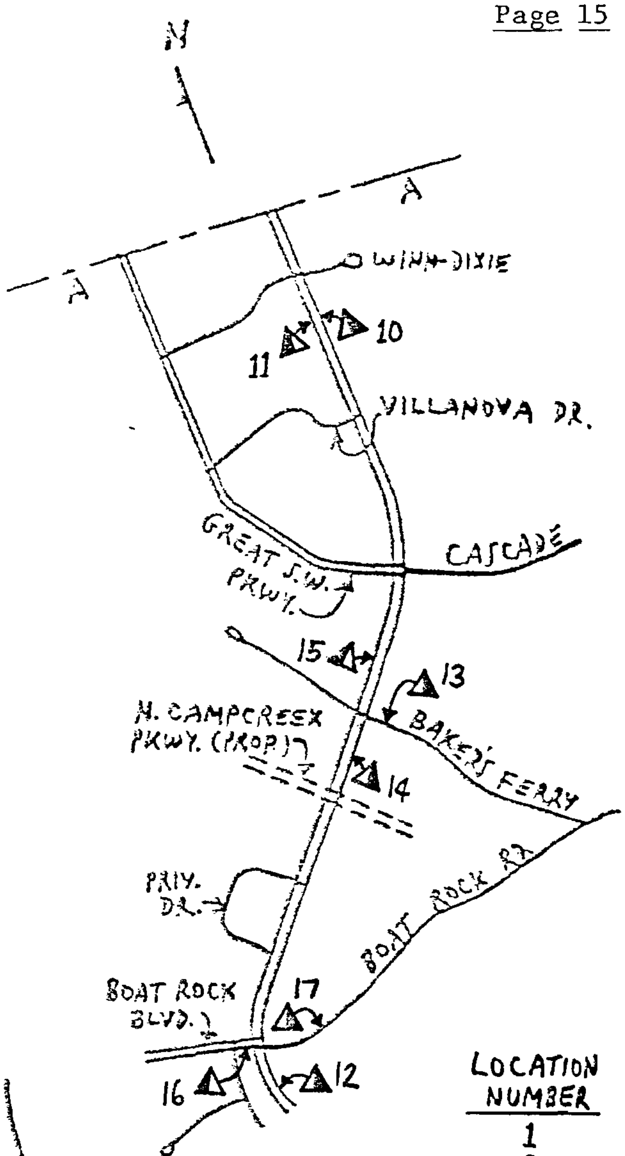
This section includes material dealing with methods of data collection. Traffic volumes, signals, signing and detector installations are discussed. Geometrics of the Boulevard are presented in updated form, relative to access point locations, the roadway cross section and adjacent land use. Mention is also made of collision data, approach speeds, and the literature survey. Additional data concerning traffic volume counts, existing signal timing, signing and literature reviewed appears in Appendices A, B, C, E and F. The plan sheets submitted with this report (36" x 24") supplement the text relative to geometrics, signing, collision diagrams and land use.

Traffic Volume Inventory

Traffic demand is one useful criterion for evaluation of needs and deficiencies. The retiming or installation of signalized intersections can be studied from user volume inputs.

An inventory of traffic volumes was conducted by both mechanical and manual recorder methods. Mechanical count station locations and corresponding 24 hour, weekday volumes appear in figure 3. Mechanical recorder cams were used which produced printouts at fifteen minute intervals. Manual counts, ordinarily taken for two consecutive hours during the morning and evening peak periods, are illustrated by peak hour in figure 4. A morning peak hour extending from 7:30 AM to 8:30 AM was continuous over the study section. That is, traffic tended to reach its peak hour during the morning within a common time period at all intersections. The evening peak hour occurred between 4:15 PM and 5:15 PM south of I-20 and from 4:30 PM to 5:30 PM north of that interchange.





LOCATION NUMBER	AVG. WEEKDAY TRAFFIC
1	11800
2	11400
3	14400
4	15500
5	13100
6	13100
7	10300
8	10100
9	1350
10	5700
11	5700
12	2750
13	950
14	5050
15	4500
16	1600
17	600

FIGURE 3

AVERAGE WEEKDAY TRAFFIC

(MACHINE COUNTS)

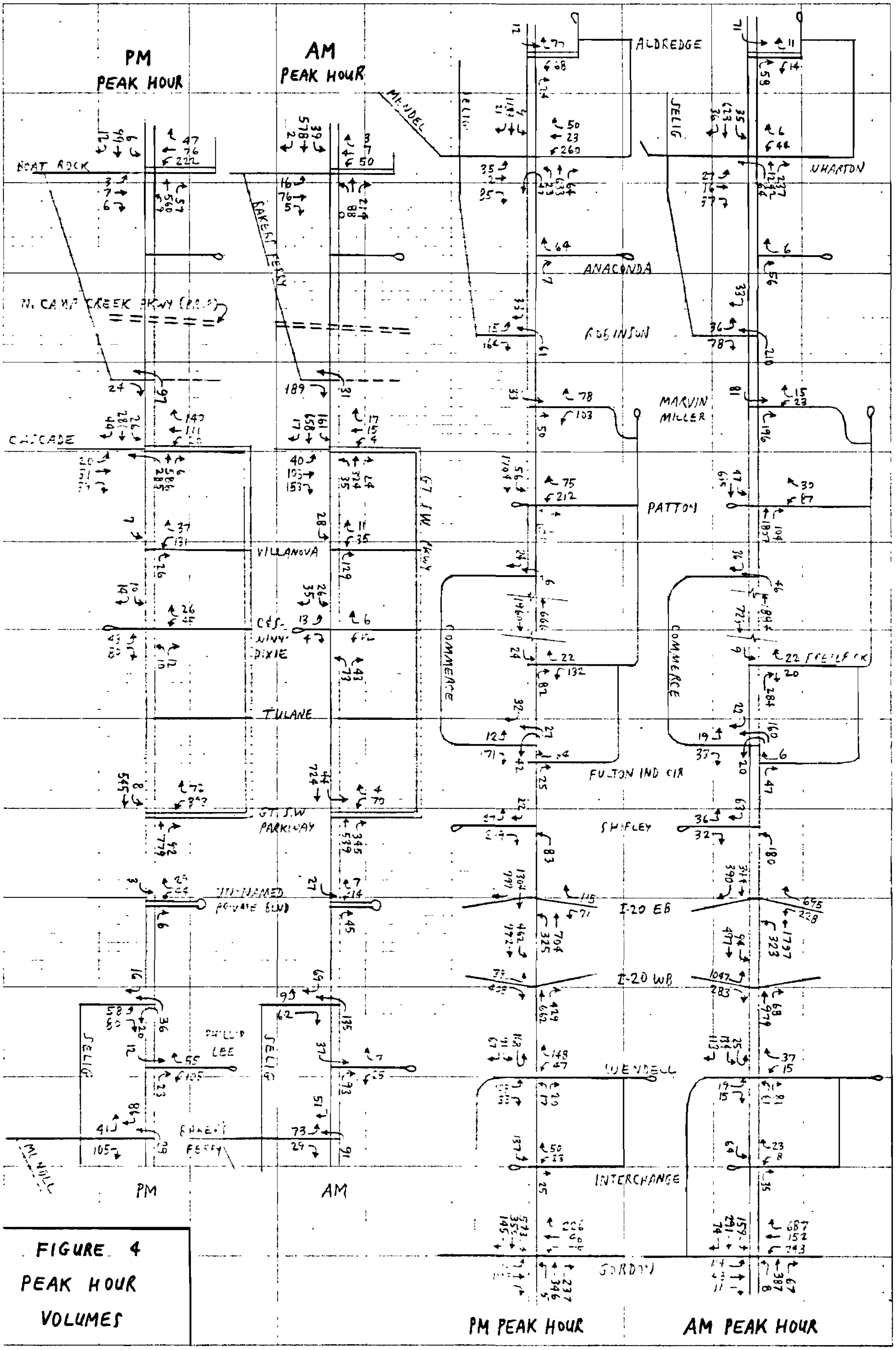


FIGURE 4
PEAK HOUR VOLUMES

A classification count was taken between the south leg of Commerce Drive and Patton Drive to determine composite truck percentages. Truck traffic was considered to be composed of vehicles having more than four tires. In addition, a breakdown of commercial delivery vehicles, single unit trucks and semi-trailers of both the gasoline and diesel variety was recorded.

Due to the lack of significant numbers of pedestrians crossing either Fulton Industrial Boulevard or any of its intersecting streets, no such counts were recorded.

Traffic Signal Inventory

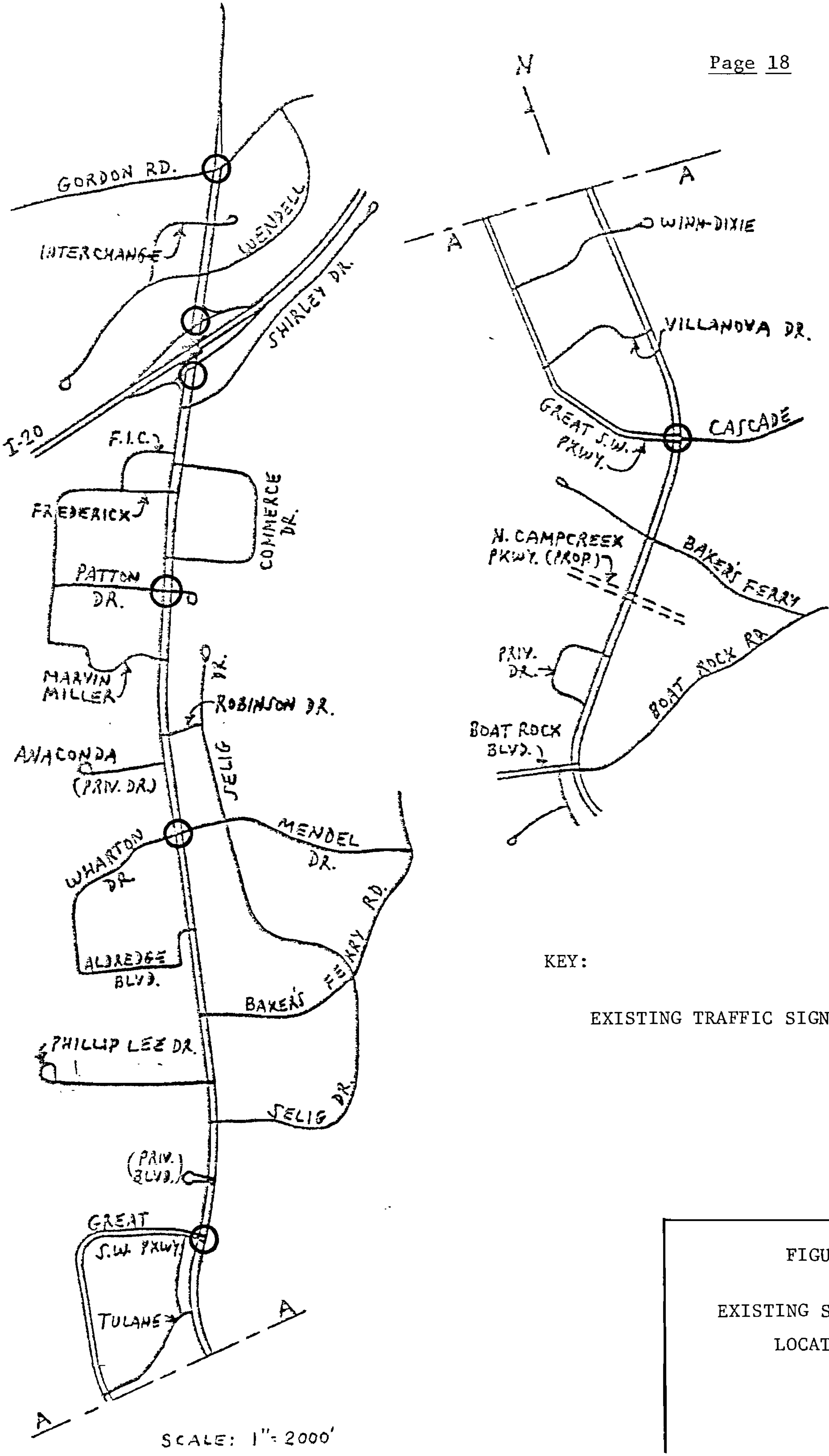
Figure 5 shows the location of signals presently installed and functioning along Fulton Industrial Boulevard. Existing timing data at the time of first inspection appears in APPENDIX C. The following is a description of each signalized intersection:

Gordon Road

An ASD 1826N(M2) controller and a pair of MM3 minor movement controllers operates this intersection on an isolated basis. All approaches have two faces, each consisting of three sections. A 12 inch red lens is used with 8 inch green and yellow indications in all cases. Both left turn bays of the Boulevard are governed by an additional signal head which contains a 12 inch circular red, an 8 inch circular yellow and an 8 inch green arrow. Phasing consists of a lead to phase A (left turns from the Boulevard), phase A, and phase B (Gordon Road).

I-20 Off Ramps

A pair of ASD 1826N(M2) controllers and a CR-16 coordinating unit control the two diamond interchange signals. Two cabinets house equipment and will be referred to hereinafter, by their relative juxtaposition to the I-20 bridge structures, as NW and SW.



KEY:

EXISTING TRAFFIC SIGNAL

FIGURE 5

EXISTING SIGNALIZED LOCATIONS

The NW cabinet houses an 1826N(M2) full-actuated controller and loop detectors for each phase. The SW houses the second 1826N(M2) controller, the CR-16 coordination unit and the associated loop detectors. At both locations three 12 inch sections, consisting of red, yellow and green circular indications, are mounted in signal heads with two faces per approach. All heads are span wire mounted except for one post-mounted signal which directs left-turn traffic to westbound Interstate route I-20.

Phasing (see existing timing sheet, Appendix C) follows an A, B, C and D, E, F isolated order at the I-20 westbound and eastbound off-ramps, respectively. Sequencing is noted in figure 6. Phases A and D regulate through traffic, phases B and E control left turns from Fulton Boulevard, while phases C and F regulate off-ramp traffic. In coordination, phasing should be capable of following any of the sequences noted in figure 7 as a function of particular demands. The ramps operate on a full-actuated basis.

Patton Drive

An ASD 807R two phase, full-actuated controller is operating on a semi-actuated basis with loops presently installed only for the side street approaches. The two-phase operation is implemented by heads containing 12 inch red, green and yellow sections. Two faces are provided per approach. An Orion 910 loop detector is utilized for the actuated approaches.

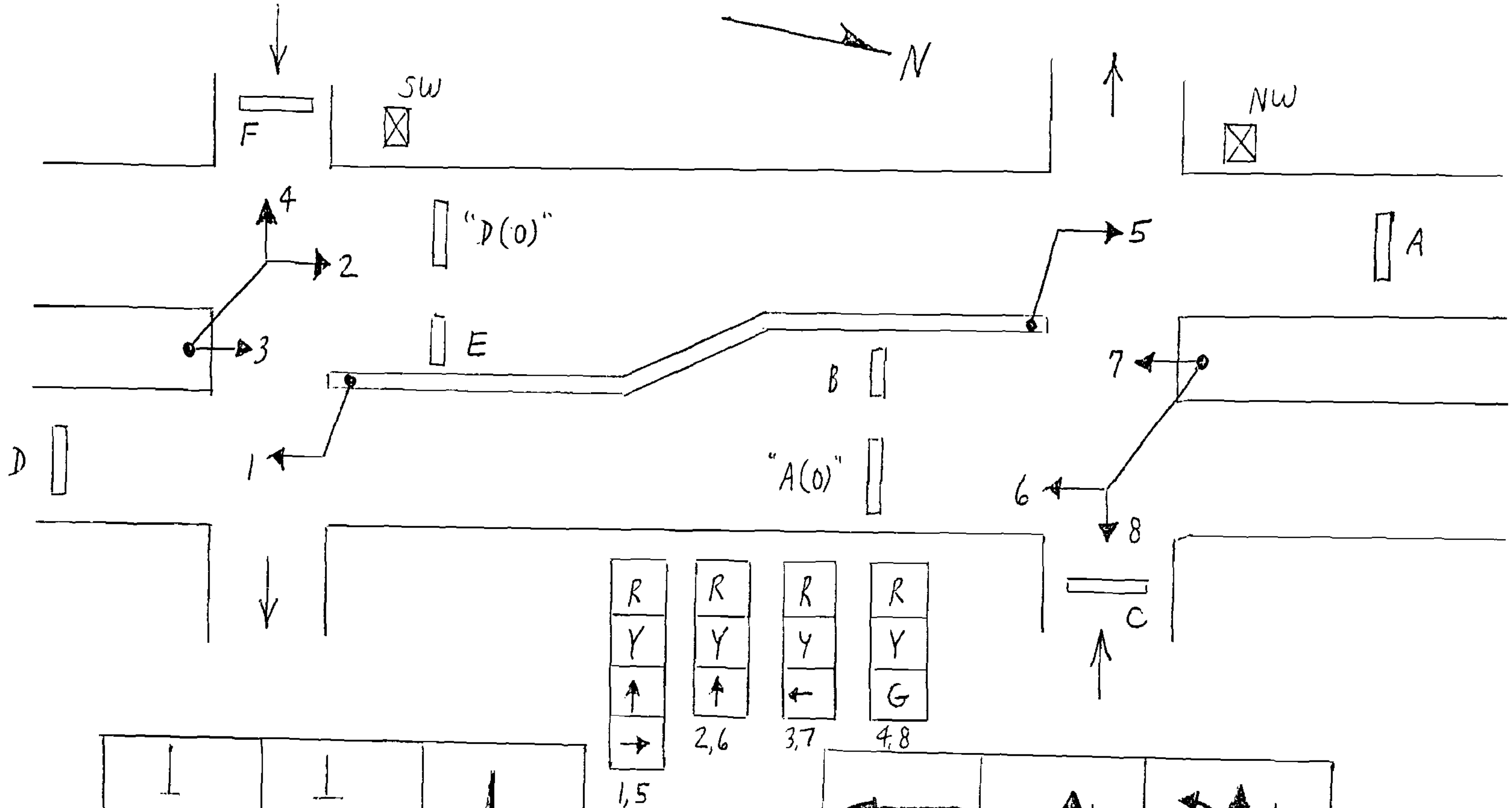
Wharton-Mendel Drives

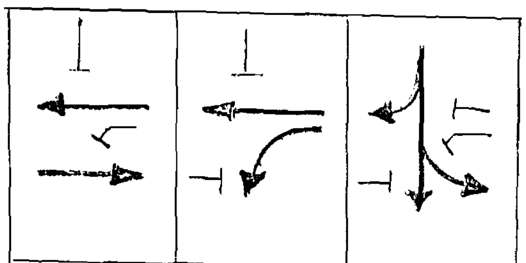
This intersection operates on a full-actuated level, with control imposed by an ASD 807R two-phase controller and a pair of Orion 910 detectors. This location operates on a two-phase basis with two signal faces per approach, each containing standard 12 inch lenses. Signals are span wire mounted, as is the case at Patton Drive.

Great Southwest Parkway

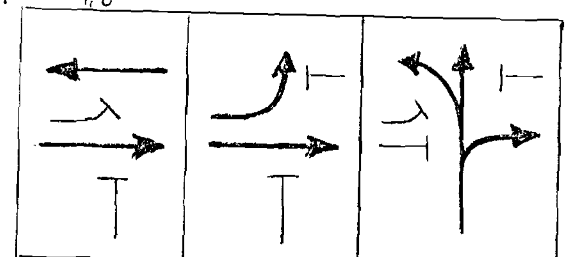
A two phase, full-actuated signal controller assigns movements at this T intersection. The controller used is an ASD 1826MF1, assisted by Orion

SOURCE: AUTOMATIC SIGNAL DIVISION





SIGNAL	D R/W	D CLR	E R/W	E CLR	F R/W	F CLR
1	↑→	Y	R	R	R	R
2	↑	↑①	↑	Y②	R	R
3	R	R	R ←	Y	R	R
4	R	R	R	R	G	Y



SIGNAL	A R/W	A CLR	B R/W	B CLR	C R/W	C CLR
5	↑→	Y	R	R	R	R
6	↑	↑①	↑	Y②	R	R
7	R	R	R ←	Y	R	R
8	R	R	R	R	G	Y

① ↑ BECOMES Y IF ØB OR ØE IS SKIPPED
② Y BECOMES ↑ IF ØC OR ØF IS SKIPPED

FIGURE 6

I-20 INTERCHANGE (AS
PHASING & DISPLAY
APPLICATION)

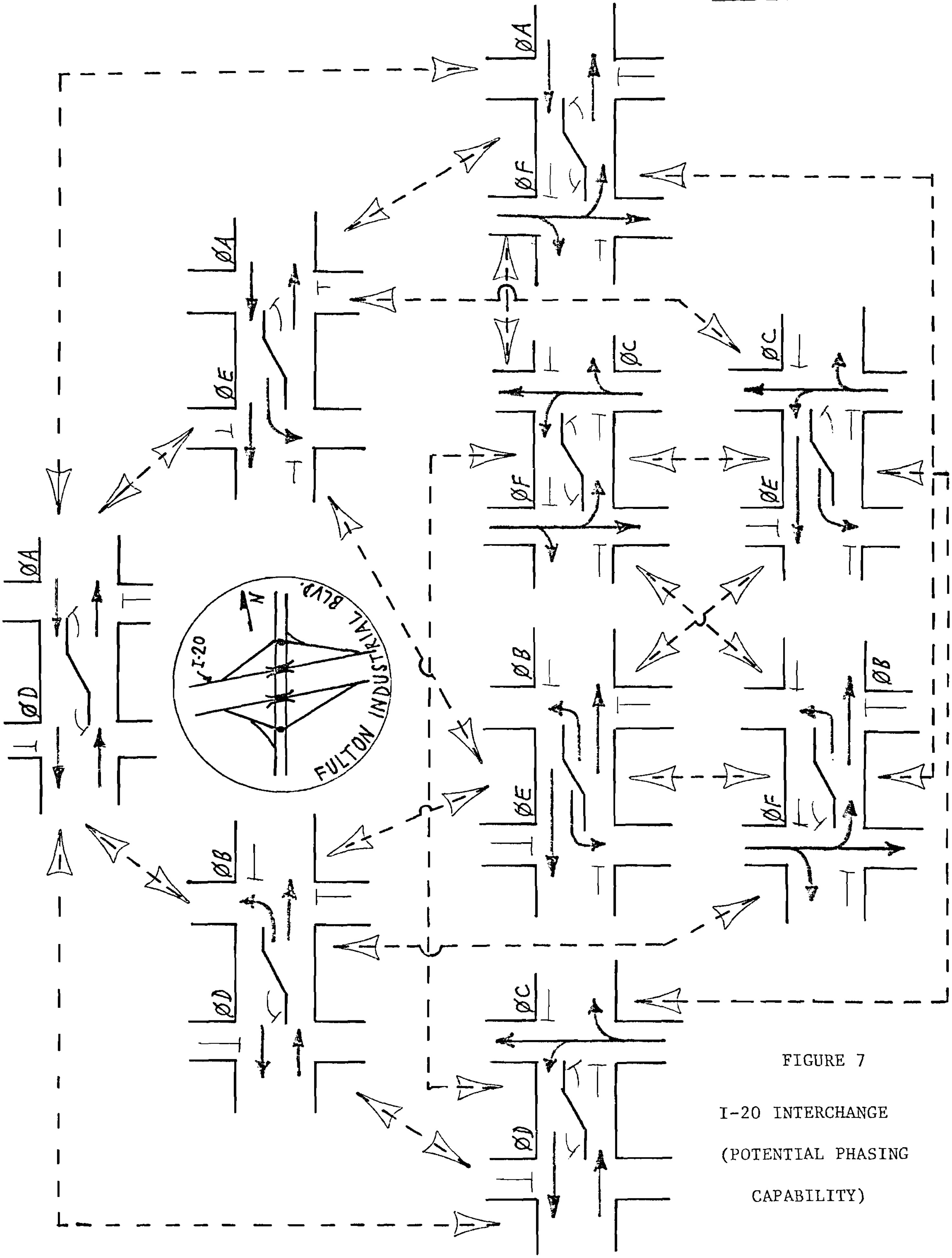


FIGURE 7

I-20 INTERCHANGE

(POTENTIAL PHASING CAPABILITY)

910 and LFE loop detectors. The installation further consists of two signal faces (12 inch lenses) per approach mounted on span wires. Turning movements into the stem of the T are so light that only two of three possible phases are presently used. The third phase can be later utilized as turning demands increase.

Cascade Road

Another ASD 1826MF1 regulates this location. Operating on a full actuated basis, this intersection requires two phases at present. The 1826MF1 type controllers are expansible to three phase operation without use of additional equipment. A pair of span wire mounted signal heads serve as indicators to each approach. Lenses are 12 inch in size for each face.

Sign Inventory

Traffic signs were noted as to designation and approximate location throughout the study area length and within 100 feet of minor street approaches. Interstate (I-20) ramps and major cross streets were surveyed at further distances to incorporate speed limits, etc. Results of the survey appear in APPENDIX E in tabular form and by sign location on the accompanying 36" x 24" plan sheets. The inventory was completed in June, 1975.

Detector Inventory

Figure 8 shows the locations of those detectors inspected during the route inventory. Those detectors serving the I-20 ramps were paved over and thus had to be approximately located by observing actuations at the controllers.

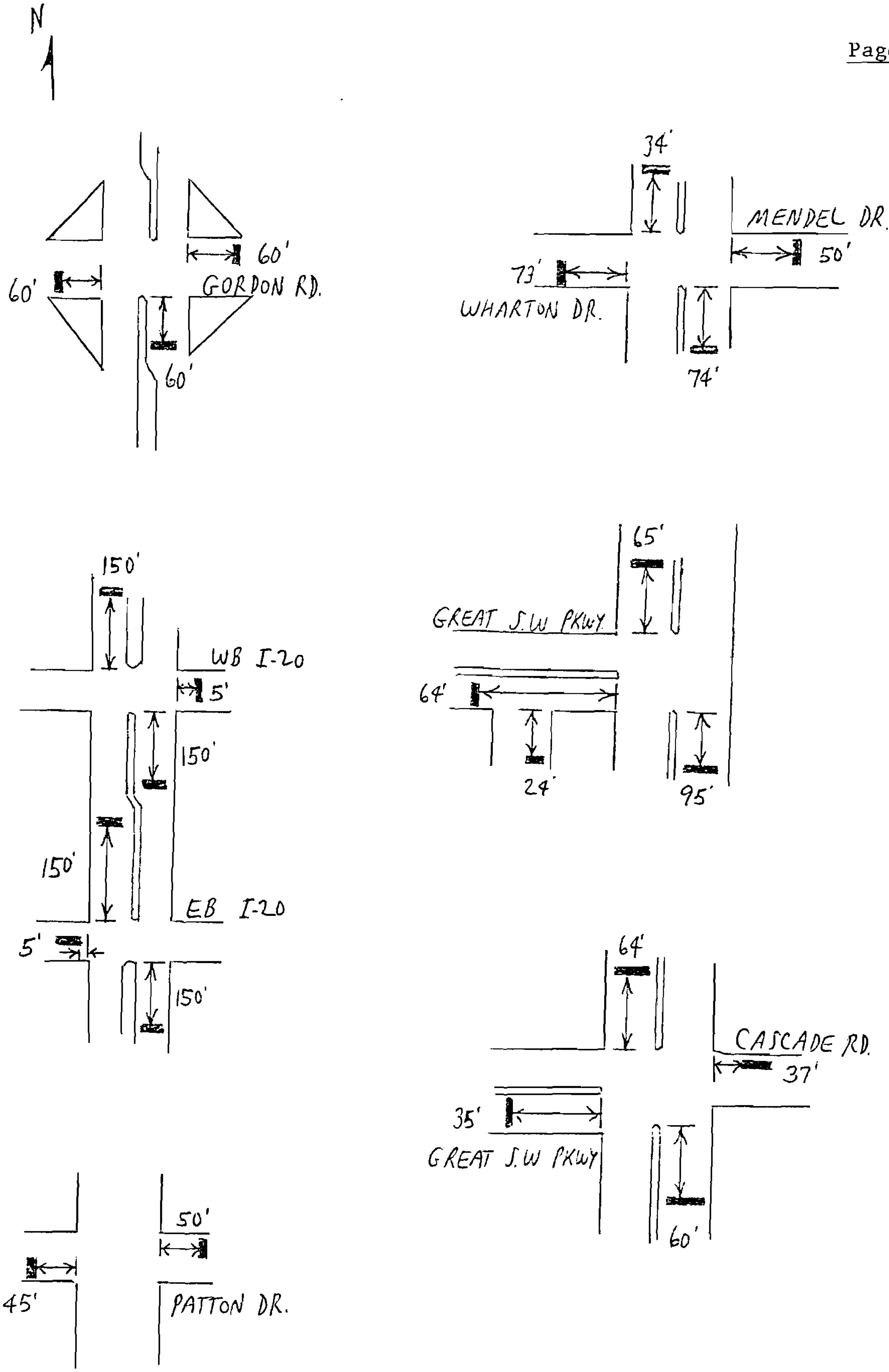


FIGURE 8: EXISTING DETECTOR LOCATIONS

Access and Roadway Inventory

Fulton Industrial Boulevard has a standard cross-section consisting of two 12-foot travel lanes in each direction, divided by a forty foot median extending from south of Boat Rock Boulevard to about 1000 feet north of Great Southwest Parkway (north leg). A ten foot oiled gravel shoulder is provided adjacent to the curb lanes in both directions. At all intersections, protected left turn lanes and exclusive right turn lanes are provided at twelve foot widths. The shoulder in the vicinity of some driveways is paved to allow deceleration and acceleration tapers.

Further north, a median width of twenty feet remains constant as far as Frederick Drive. Other roadway features in this segment are the same as those to the south.

Between Frederick Drive and Gordon Road, the median varies from fourteen to thirty-two feet. Two eleven to twelve foot travel lanes are maintained through this section. A continuous right turn lane extends south from Gordon Road to the westbound I-20 on-ramp. The lane resumes as an acceleration lane for ramp traffic entering the southbound Boulevard from eastbound I-20 just south of the interchange. This lane then continues as a right turn lane to Frederick Drive. In the northerly direction, during off peak hours, this shoulder is used as a through lane on occasion as far south as Patton Drive. During the evening peak period this shoulder is regularly utilized by through traffic generally with right turn destinations to I-20 via the eastbound on-ramp. North of the interchange the shoulder is paved to function as a right turn lane to Wendell Drive. Between Wendell Drive and Gordon Road, the standard northbound section consists of two travel lanes and a ten foot shoulder.

The 36" x 24" plan sheets submitted with the report function to update original construction plans used as a base. New intersections and driveways have been added, while modifications to the interchange and other locations are included.

Table 1 indicates distances between existing cross streets and major driveways throughout the study area. Distances were measured between the striped centerline of the traveled ways.

Inventory of Speeds

The Boulevard is posted for 55 mph south of Great Southwest Parkway (N. leg) and 50 mph as far north as Gordon Road. The southbound approach to Gordon Road is posted at 40 mph. Gordon Road traffic is regulated by speed limits of 35 mph and 45 mph for the westbound and eastbound approaches, respectively. Interstate I-20 ramp traffic is posted at 35 mph for the westbound approach to the Boulevard, while the eastbound off-ramp receives an indication of 30 mph. Bakers Ferry Road (N.leg) has a posted approach speed of 35 mph and Boat Rock Road has a 40 mph limit. Other approaches are not posted. Using an Argo-Kienzle tachograph, speed and delay runs were conducted over the length of the study section on Tuesday, July 8 (7:30 am - 8:30 am) and on Thursday, July 10 (4:15 pm - 5:30 pm). Table 2 summarizes results averaged over all runs. The Thursday runs were conducted during a period of unusually light traffic.

Accident Inventory

Data collection was accomplished through review of Atlanta Police Department accident reports for Fulton County. Information recorded, as available, included location, date, day of week, time of day, road condition, type of accident, number of accidents involving personal injury and collision diagrams.

Evaluation of accident data appears in the ANALYSES section of this report for 1974.

TABLE 1

ACCESS CONTROLS

Intersection	Spacing to next intersection (feet)	Present Access[1]
Gordon Rd.	930	4 Way
Interchange Dr.	659	4 Way[2]
Wendell Dr.	703	4 Way
I-20 WB ramp	656	"T"
I-20 EB ramp	510	"T"
Shirley Dr.	822	"T"
Commerce Dr. (N. leg)	409	"T"
Frederick Dr.	958	"T"
Commerce Dr. (S. leg)	462	"T"
Patton Dr.	920	4 Way
Marvin Miller Dr.	1181	"T"
Robinson Dr.	345	"T"
Un-named Dr. (Anaconda)	1030	None
Wharton-Mendell Drs.	1373	4 Way
Aldredge Blvd.	1142	"T"
Bakers Ferry Rd. (N. leg)	1058	"T"
Phillip Lee Dr.	489	"T"
Selig Dr.	703	"T"
Un-named Blvd. (Auto-Solar)	999	"T"
Great Southwest Pkwy (N. leg)	950	"T"
Tulane Dr.	1581	None
Un-named Dr. (Winn-Dixie)	1370	4 Way
Villanova Dr.	1620	"T"
Cascade Rd.	1478	4 Way
Bakers Ferry Rd. (S. leg)	830	4 Way
(N. Campcreek Pkwy.)	947	4 Way at Grade[3]
Un-named Dr. (Factory)	1755	"T"
Boat Rock Blvd. and Rd.		4 Way
	25880	
	(4.9 Miles)	

[1]Via median opening

[2]Considering Day's Inn driveway

[3]Proposed by others (1)

TABLE 2

EXISTING SPEEDS[1]

Intersection	AM PEAK HOUR				PM PEAK HOUR			
	NB[2]	SB[2]	NB[3]	SB[3]	NB[2]	SB[2]	NB[3]	SB[3]
Gordon Rd.								
	36	27	36	19	30	30	18	16
WB I-20								
	30	26	10	20	32	32	32	32
EB I-20								
	39	33	14	33	31	40	21	40
Commerce Dr.								
	47	34	47	27	31	40	31	40
Patton Dr.								
	47	36	47	32	42	40	32	40
Wharton Dr.								
	46	49	46	49	40	42	34	42
Bakers Ferry Rd. (N. leg)								
	46	51	46	51	39	41	39	41
Gt. S.W. Pkwy (N. leg)								
	54	52	42	52	39	43	26	43
Winn Dixie Dr.								
	44	48	44	44	41	42	41	42
Cascade Rd.								
	50	50	37	50	43	50	32	50
Un-named Dr.								
	53	43	53	43	42	40	42	40
Boat Rock Blvd.								

(1) In miles per hour

(2) Average running speed

(3) Overall travel speed

Land Use Inventory

Often, the nature of adjacent development can be one criterion which establishes the characterictics of a traffic artery. This is the case along Fulton Boulevard which carries a heavy flow of commercial traffic circulating between Interstate I-20 and the numerous industrial sites bordering the Boulevard. The series of 36" x 24" maps, which supplement this report, depict land uses immediately adjacent to the through-route. Generally, the section of Fulton Boulevard south of Boat Rock Road is largely underdeveloped, at present, while sporadic development occurs between Boat Rock Road and Wharton-Mendell Drives. To the north, as far as Gordon Road, adjacent acreage is generally established for service, commercial and industrial uses.

Literature Survey

Using the Georgia Institute of Technology Information Exchange Center, various data bases were periodically reviewed to determine the most recent policy and standards guidelines with regard to access control and median opening spacing. Data sources utilized included; the Civil Engineering Combined Index (ASCE) and abstracts of various highway and traffic engineering research publications. Numerous Highway Research Records were reviewed for both access control and traffic signal information. The computerized literature search in all cases was retrospective to January 1, 1969. Results of the survey appear in Appendix F.

SECTION II

-ANALYSES-

ANALYSES

This section deals with the development of concepts and standards to solve traffic problems along the Boulevard. A volume/capacity study includes consideration of traffic growth trends, vehicle classification and particular capacity deficiencies. Appendix D contains volume/capacity calculations. Traffic signal warrants, locations and display types (optically-programmed) are discussed with reference to particular sites requiring attention. Signal phasing and storage are examined from applications to at-grade intersections, where the merits of leading and lagging greens are summarized. Diamond interchange phasing sequences are studied to arrive at an optimal pattern for the I-20 ramps. Analysis of the CR-16 coordinating unit currently used at this location is also presented. Detector selection and location to avoid accident situations are covered. A traffic adjusted systems concept is examined for future application to Boulevard intersections. Access guidelines for both signalized and un-signalized intersections are reviewed to develop basic policies. Critical accident sites are defined and analyzed.

Analysis of Volumes and Capacity

Traffic along the Boulevard generally follows a telescoping pattern in which heavy volumes in the I-20 interchange vicinity progressively decrease to the south (toward Boat Rock Boulevard) for both directions of travel.

Traffic Trends

Projected increases in volume can directly influence design considerations by indicating the need for future improvements which may not be presently required based on existing demand. The Georgia Department of Transportation counts taken in 1970 at the I-20 interchange were used as a base for comparison of 1975 traffic data. A straight line projection indicates that by 1980 the number of vehicles entering the interchange will increase by 32 percent over 1975 counts. Of greater interest is how this increase is apportioned by direction from the interchange. During the morning peak hour, traffic in the critical (SB) direction is anticipated to grow by a negligible amount in the section between I-20 and Gordon Road. However, an increase of 45 percent is predicted for the southbound segment south of I-20. In the evening peak period, increases of 10 percent and 24 percent are predicted for the respective northbound segments. Figure 9 documents specific data at the I-20 interchange by direction for both 1970 and 1975.

A factor was calculated to determine the proportion of each peak hour as a percentage of the ADT for side street approaches.

Values of 0.20 and 0.04 resulted for the morning and evening peak hours, respectively. These figures were used as guides to estimate average weekday traffic at side streets and driveways where 24 hour machine counts were not taken (eg. Commerce and Frederick Drives). Volumes thus estimated could determine warrants for signalization.

Classification count data indicated that about 16 percent of the vehicles using the Boulevard are trucks. Particular percentages by direction and peak hour were determined for inclusion in the capacity analysis (see Appendix D).

Volume-capacity ratios were calculated for each of the existing signalized intersections as well as for Boat Rock Boulevard. Storage capacity of left turn lanes was also checked where applicable.

Capacity & Storage Deficiencies

Left turn traffic from northbound Fulton Boulevard to westbound Gordon Road extends beyond its turn bay during the evening peak hour. This creates accident hazards with through traffic in the high speed lane.

A level of service "D" is experienced at 120 second cycle lengths during both the morning and evening peak hours at the northern I-20 intersection.

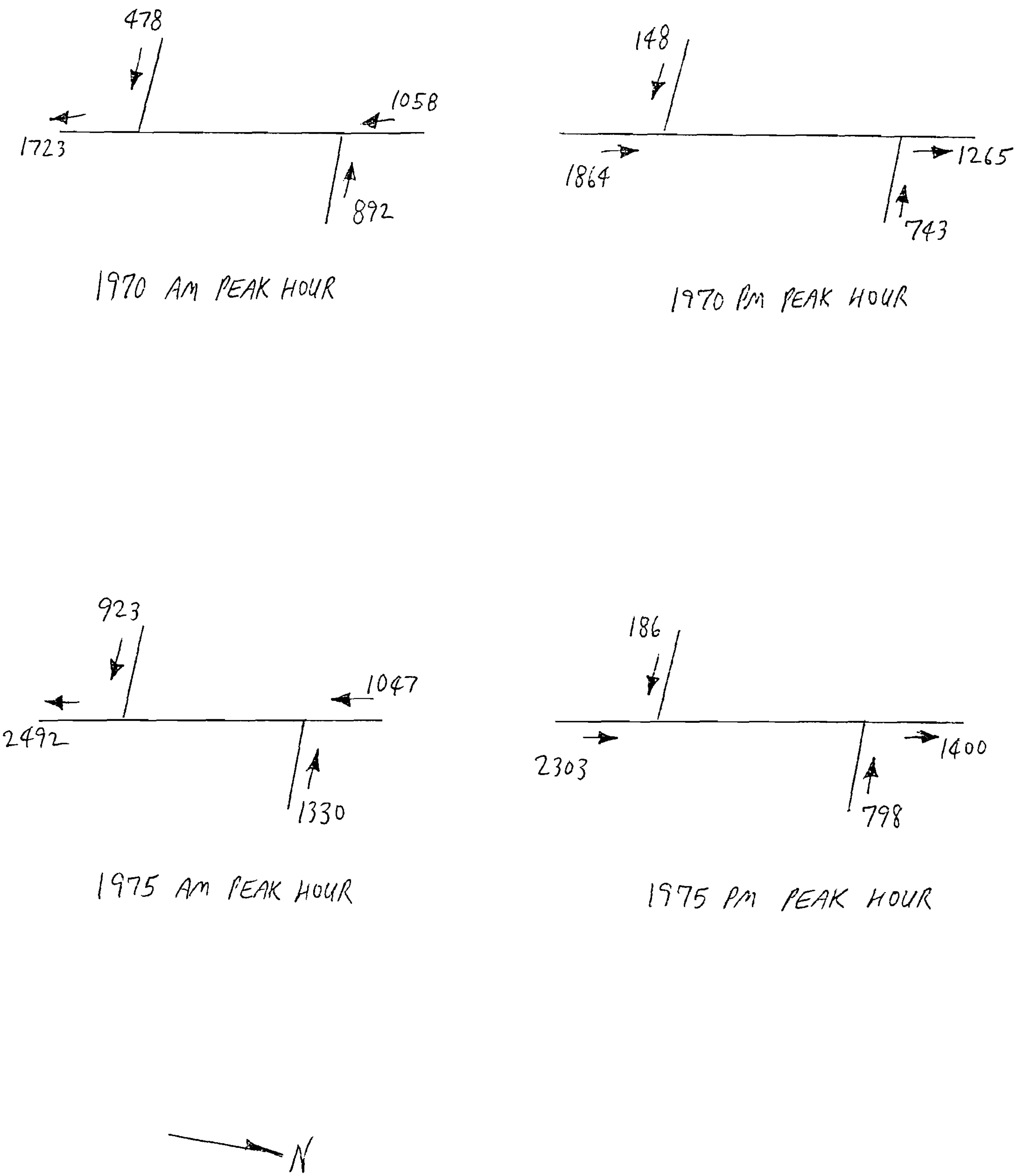


FIGURE 9

VOLUME TRENDS (FULTON IND. BLVD. AT I-20)

This service level is considered to be substandard. During the AM peak hour the southern I-20 intersection operates at a level a level of service "D" at a 120 second cycle length. Conditions worsen during the PM peak hour, with a level of service "E" predicted, assuming coordination within the interchange. Level of service "E" refers to a traffic condition in which unstable congestion occurs, resulting in intolerable delays. A volume/capacity ratio equal to, or nearly, 1.0 is representative of this condition. Without proper coordination,jammed conditions have been consistently observed. Both left turn lanes leading from the Boulevard, to I-20 on-ramps, are of inadequate length. Storage requirements, based on 120 second cycles and design chart calculations (2), indicate a need for 640 feet for the northbound to west-bound movement and 425 feet for the opposing (SB) Boulevard turning maneuver to eastbound I-20.

Patton Drive is constricted by its single eastbound approach lane which reduces this intersection's level of service during the PM peak hour to "E".

The Southbound left turn lane at Cascade Road was observed to be exceeded, upon occasion, by stored traffic waiting to proceed east during the evening peak hour.

Analysis of Signal Warrants

Existing or proposed signal installations should be justified by meeting warrants prescribed by the Manual on Uniform Traffic Control Devices. The Manual also specifies signal displays required for general conditions (3).

Prior to direct application of signal warrants, a comprehensive study of traffic and physical characteristics unique to the location should be investigated. Such data should consist of: a vehicular count taken over the heaviest 16 consecutive hours of a 24 hour period, and a 15 minute interval count during the peak two hour morning and evening periods. A traffic control base plan should indicate intersection geometrics, channelization, and other pertinent data.

At least one of the Manual warrants must be met to justify signal installation. When a signal is warranted by the Manual, it and related traffic control devices should be located and installed according to Manual standards.

The warrants which may be met include the following:

1) Minimum Vehicular Volume

2) Interruption of Continuous Traffic

3) Minimum Pedestrian Volume

4) School Crossing

5) Progressive Movement

6) Accident Experience

7) Systems Warrant

8) Combination of Warrants

All presently signalized intersections along the Boulevard meet at least one of the above warrants.

Table 3 depicts a conversion of warrants number 1 and 2 on an ADT (vs. hourly) basis as developed by Paul Box and Associates (4).

Review of presently unsignalized locations indicates that none meet warrants number 1 or 2 based on 1975 traffic volumes. Warrants number 3 and 4 do not presently, and are unlikely in the future, to apply to any study area intersection. Progressive movement control may be used to justify signal installations at locations where they are not otherwise warranted, in order to maintain proper platooning of vehicles and effectively regulate platoon speed (5). Signals installed under this warrant should be further restricted to spacings which conform to feasible access standards (see <u>Access Guidelines-Analysis of Literature</u>).

TABLE 5

VOLUME REQUIREMENTS

FOR

SIGNAL WARRANTS

Number of lanes for moving traffic on each approach		Vehicles per hour on major street (total of both approaches)	Equivalent A.D.T.	Vehicles per hour on higher volume minor street approach (one direction only)	Equivalent A.D.T.*
MAJOR STREET	MINOR STREET				
1	1	500	8,300	150	4,600
2 or more	1	600	10,000	150	4,600
2 or more	2 or more	600	10,000	200	6,000
1	2 or more	500	8,300	200	6,000
1	1	750	12,500	75	2,300
2 or more	1	900	15,000	75	2,300
2 or more	2 or more	900	15,000	100	3,100
1	2 or more	750	12,500	100	3,100

*Box, P. Warrants for Traffic Control Signals, Traffic Engineering, November, 1967

Notes:
1. Minor street ADT of 3600 (one-lane) and 4800 (two lane) have been accepted in some instances as meeting Warrant I requirements.
2. For one-way minor street reduce minor street ADT requirements by 33 1/3%

The accident warrant requires, in part, that at least five collisions susceptible to correction by signalization to have occurred within a one year period. Two intersections, Frederick Drive and Commerce Drive (S. leg), are likely to meet conditions satisfying the Accident Experience warrant within the next five years. The Systems Warrant could be satisfied at Bakers Ferry Road (N. leg) if a system were to be developed which included this location.

Analysis of Signal Location

The Manual on Uniform Traffic Control Devices prescribes the use of a minimum of two signal faces per approach for through-traffic. A single signal face can be used for exclusive turning lanes.

Sight distance is a critical concern in placement of signal faces. As speeds increase,a greater minimum visibility requirement must either be met or advance warnings of the signal be indicated.

On any given approach, signal heads should be within a visual cone described by a trapezoidal plan section. From a point bisecting the stop line, a visual range should be extended 20° either side of a line perpendicular to the stop line. The trapezoid of application is included between 40 and 120 feet from the stop line point. Figure 10 illustrates this concept.

Other criteria to be followed include: an eight foot horizontal separation between signal heads for any approach and the use of a supplemental near side signal, when the 120 foot limit must be exceeded due to intersection geometry.

The Manual also denotes standards for the mounting height of signals, display of lenses with regard to arrangement and number, and application of signal indications.

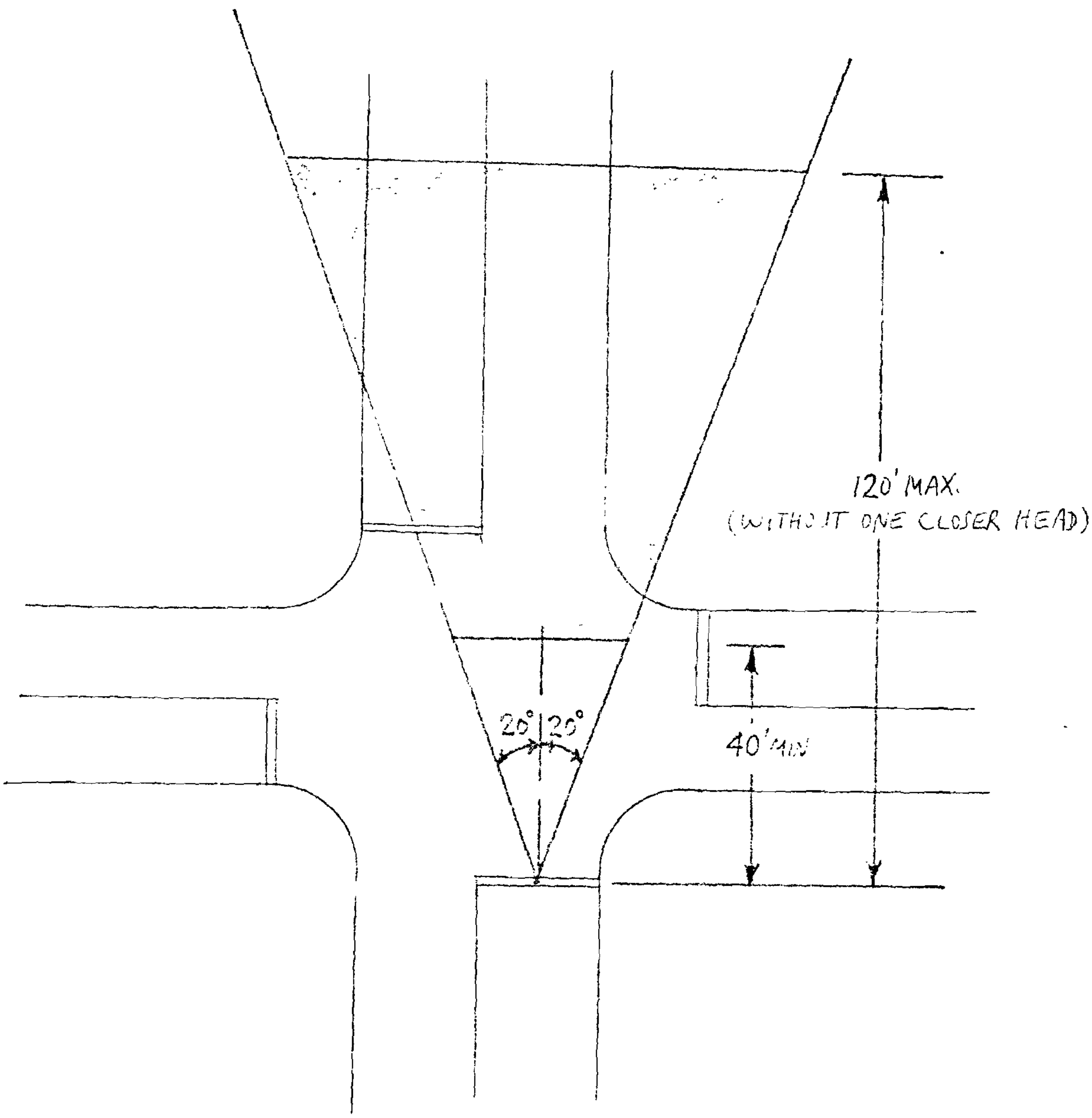


FIGURE 10

CONE OF VISION

Location of signals along the Boulevard conforms to MUTCD requirements, with the exception of Patton Drive's westbound approach. The signal heads are considerably less than eight feet apart for drivers using this approach.

Optical Displays

The 3-M information booklet for design of optically programed signals was used to consider applications of special signal displays where required, such as at the I-20 interchange.

Studies of driver characteristics at signalized intersections have provided information concerning perception times, reaction times, judgement factors, and vehicle performance. Such studies have shown that intersection capacity is reduced and accident frequencies are increased by driver confusion resulting from apparently conflicting signal indications. Optically programed signals use selective as well as adjustable restriction of signal visibility to reduce such conflicts.

When irregular intersection design requires placing signals for different approaches with a small angle between indications, it is suggested that each indication be designed so an approaching driver can see only those signals controlling his (her) approach.

Optically programed signals limit visibility of an indication exclusively to the roadway area where it must be seen. This is accomplished by an optical system in the signal head.

Warrants have been developed by the 3-M Company for using such signals for intersections which are closely spaced, are skewed or have exclusive turning movements (6).

One location, the intersection of the westbound I-20 off-ramp, has been the scene of three accidents in which motorists have mistaken the signals controlling another approach for their own. It is advised that optically programmed signal heads be used at this location (see the Recommendations section).

Signal Phasing and Storage

The intersection at Fulton Industrial Boulevard and I-20 is an example of a conventional type diamond interchange. Other existing intersections are of the non-interchange variety. Phasing, sequencing and storage will be treated both for diamond interchanges and for at-grade intersections.

Objectives of signal timing include keeping the number of phases to a minimum, the use of short cycle lengths when capacity is not a concern, and the use of longer cycles during periods of heavy demand. However, cycles longer than necessary to accomodate traffic present will produce higher average delays. Therefore, the shortest cycle should be selected that will accomodate the demand present and thus will produce the lowest average delay.

Prior to discussing some concepts and practical applications of phasing and storage, an analysis of the CR-16 coordinating unit presently in place at the I-20 interchange is in order.

Operation of the CR-16 Coordination Unit

The CR-16 coordinating unit has one application in coordinating a pair of closely spaced intersections having a common two-way artery. At the I-20 ramps, a CR-16 has been combined with two three-phase 1826N(M2) controllers to allow the potential of complete skipability (see figure 6).

Two external controls, timer 1 and timer 2 allow adjustment of the CR-16 to particular intersection characteristics. For example in figure 11, timer 1 functions to (1) hold and (2) time. The hold function of timer 1

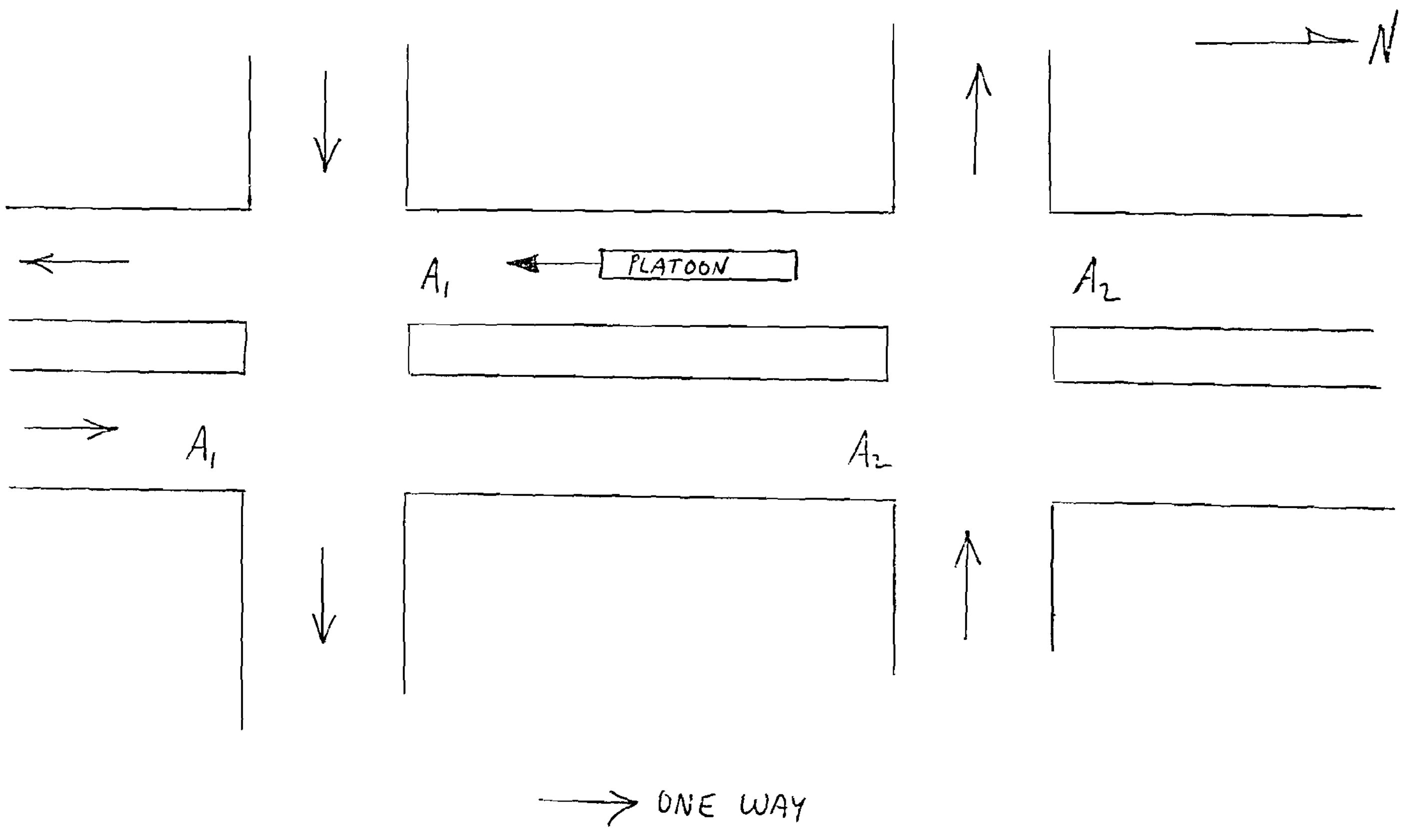


FIGURE 11

OPERATION OF THE CR-16 COORDINATING UNIT occurs whenever one controller is in phase A_1 and the other is calling to go to phase A_2. The one already in phase A_1 is held in that phase until the other begins phase A_2. The timing function of timer 1 results from its timing a continuation of the hold, after phase A_2 begins, until the platoon has had time to reach the phase A_1 detectors and extend the green by detector actuations.

Since the platoon has left intersection 2, phase A_2 gaps out and this intersection goes to phase B_2 or phase C_2. Meanwhile, the platoon is moving through intersection 1 on the phase A_1 green. If a call should occur at intersection 2 for the green to return to phase A_2, a problem results since timer 1 will attempt to re-enter its hold function. Thus, a cyclic situation will occur in which certain sequences are repeated at the exclusion of other demands. Clearly, a need exists to be able to override or disable timer 1 so that intersection 1, in this case, can leave phase A_1. This overriding or disabling effect needs to be established in the form of a permissive period at intersection 1.

Timer 2 begins to time when timer 1 has finished and phase A_2 has gapped out and gone to phase B_2 or phase C_2. Timer 2 times for intersection 1 a permissive yield period. This period must be of sufficient length for intersection 1 to max out, because a premissive period cannot be utilized if the controller is still timing extensions on phase A_1. If Timer 2 is set too low, the permissive period will pass with intersection 1 remaining in phase A_1. In this case Timer 1 will continue to hold intersection 1 in phase A_1 until intersection 2 goes to phase A_2, and therefore the restrictions will be recycled. If Timer 2 is set too high, it will still be timing when intersection 2 returns to phase A_2.

At-Grade Intersection Applications

Although some of the principles applied to at-grade phasing could also pertain to the diamond interchange, they are discussed here for clarification before consideration of the more complex interchange problem.

The number of phases utilized at any location depends on the composition and direction of traffic flow as well as the number of intersection approaches. In determining the number of phases, additional delays are added and capacity is reduced as additional phases are included . Where special conditions exist, such as heavy turning movements, added phase requirements may be justified.

Certain numbers of phases are usually associated with specific at-grade operations. At four-legged intersections, similar to some of those on the Boulevard, two phases will tend to provide the most efficient operation. Inclusion of heavy turning movements may dictate the need for a third phase, as is presently the case at both Gordon Road and the I-20 ramps. When an unbalanced flow, by direction, is prevalent, a split phase (leading or lagging green) may be employed.

A number of advantages and disadvantages are associated with both leading and lagging (or advanced and trailing) greens. Specifically figure 12 illustrates these two cases.

In the case of leading green (left turn) movements, the following observations can be made (7).

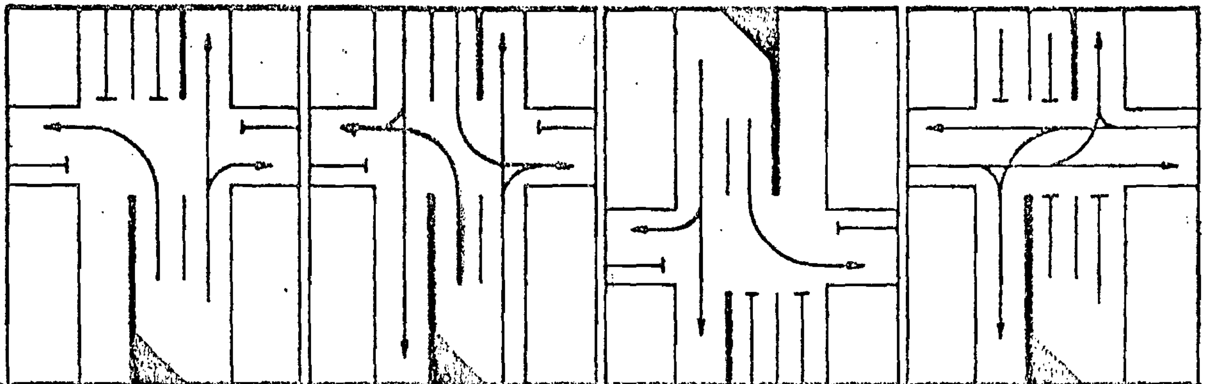
Advantages:

(1) Allows a lighter left turn to terminate early so opposing thru-traffic will receive more green time.

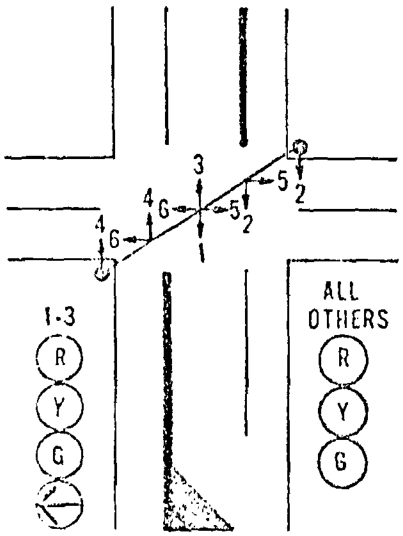
(2) Reduction in congestion by moving turns initially in the cycle.

(3) When opposing thru-traffic is light, encourages left turns on the circular green indication.

ADVANCE AND LAGGING LEFT TURN (FIGURE 12)



SIG-NAL	ADVANCE A		PHASE A			LAGGING A		PHASE B		
1	G	G	G	Y	R	R	R	R	R	R
2	G	G	G	Y	R	R	R	R	R	R
3	R	R	G	G	G	G	Y	R	R	R
4	R	R	G	G	G	G	Y	R	R	R
5	R	R	R	R	R	R	R	G	Y	R
6	R	R	R	R	R	R	R	G	Y	R
		*			*					**



SIGNALS

* MANDATORY
** OPTIONAL

SOURCE: FEDERAL HIGHWAY ADMINISTRATION, OFFICE OF TRAFFIC OPERATIONS

(4) Requires a single clearance at the end of the straight-through green phase.

Disadvantages:

(1) Creates vehicle/pedestrian conflict during the advance interval.

(2) If turning movement gaps out early, the opposing through-traffic platoon may start too early (ie. progression will be affected).

The lagging green is evaluated as follows:

Advantages:

(1) Approximates normal driver behavior.

(2) Eliminates pedestian/vehicular conflicts.

(3) Provides accurate progression.

(4) Allows an overlap of applicable right turns from the minor street with the left turn off the main street.

(5) Drops the left turn call, if the turning movement is capable of being previously made on the circular green.

Disadvantages:

(1) The overlap of opposing left turns can create wasted green time when one movement is considerably lighter.

(2) Creates an obstruction to through-traffic when an exclusive turn lane is not provided.

(3) Where there is no actuated turn, the tendency is to wait for a green arrow rather than moving on the circular green when gaps in opposing traffic occur.

(4) Driver expectation may be violated by the use of a lagging green at a high-speed four legged intersection. This occurs when motorists mistakenly assume that both

directions are being stopped at the same time.

(5) The left-turn slot must be of sufficient length to store the expected queue.

Channelization geometry can be occasionally used to provide exclusive movement for certain travel desires and thus reduce the number of phases required. An example occurs in the case of "jug-handle." The jug-handle is one method to provide a two phase operation when the number of left turns desiring to enter the stem of a T intersection becomes excessive. The Highway Capacity Manual illustrates this concept on page 328. As traffic increases on the Boulevard, the use of jug-handles may become warranted.

Diamond Interchange Applications

An extensive signal system is required at diamond interchanges because of the short spacing between the two signalized intersections. A variety of potential traffic maneuvers also complicates the procedure. The following paragraphs are taken from the "Operational Study of Signalized Diamond Interchanges" developed by the Texas Transportation Institute (8).

Signalization should serve to separate all high-volume conflicting movements in the interchange area and should minimize vehicle storage between the two intersections.

Figure 13 shows a pair of possible phasing arrangements for diamond interchange signalization.

Sequence I requires that clearance phases be added following phases A and B to clear interior approaches. This clearance allows storage room for the ramp movement on phase C. This sequence creates a four-phase cycle (if the two clearance intervals are considered approximately equal to one phase) with a considerable waste of time. Another disadvantage of Sequence I is the sluggish operation frequently encountered on the phase A movement. This results from residual storage on the interior approaches during phase C.

SOURCE; "TRAFFIC ENGINEERING," MARCH, 1966.

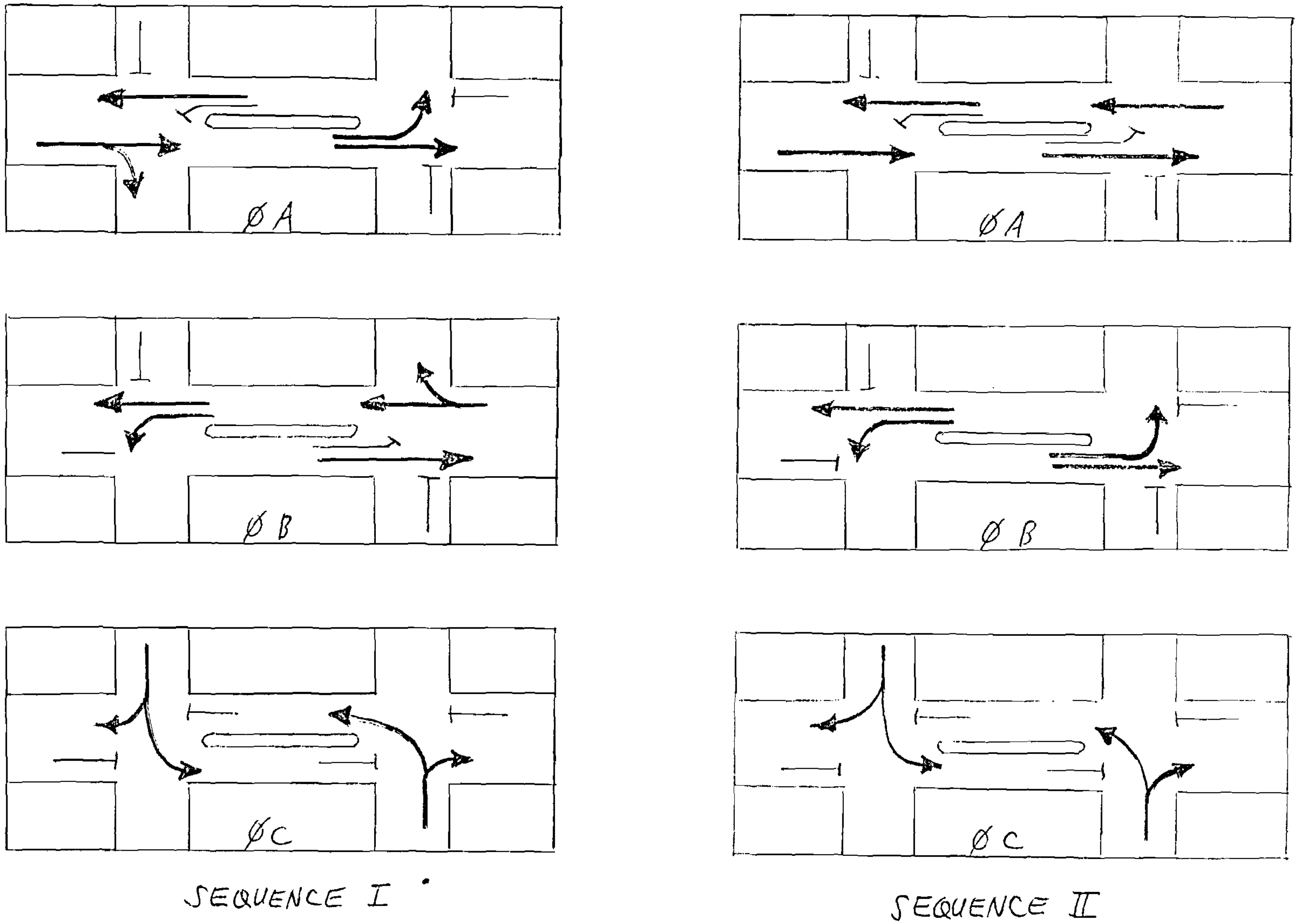


FIGURE 13

DIAMOND (CONVENTIONAL PHASING)

A final problem of the first sequence is a limitation on capacity for the ramp movements. The number of vehicles that can be moved from a ramp approach during phase C is governed by the storage capability of the interior approaches. Therefore, this sequence is not adequate to accomodate large ramp movements.

Sequence II allows interior approaches to clear on phase B and give preference to major street movements. A serious left-turn storage problem can, however, be created. Left turns from both of the main street approaches are stored during phase A. When a heavy left-turn demand from a main street approach occurs, the storage capacity for left-turning vehicles is exceeded and intersection blockage results. Sluggish operation will follow phase C, and storage limitations will exist for the ramp movements as in sequence I.

Other three-phase arrangements similar to the sequence previously discussed yield the same basic problems of inadequate storage and inefficient operation.

A four-phase system with overlaps (see figure 14) was developed to eliminate these problems. Each of the four approaches is given a separate phase and is permitted to move through the entire system upon receiving a green indication. This eliminates storage capacity limitations that develop on interior approaches. Consideration of each movement in the four-phase sequence shows that vehicle storage on the interior approaches is practically eliminated. The only vehicles requiring storage are those making a U-turn movement from a ramp during the last seconds of a phase. This seldom stores more than 2 vehicles per cycle and has little effect on operation. Thus, the left turn storage problem is eliminated.

An additional advantage of the recommended phasing is the efficiency that can be obtained. An overlap of the ramp and major street phases (phases A and C overlap) is possible due to starting delay and travel time of the major street traffic in moving from one intersection to the other.

SOURCE: "TRAFFIC ENGINEERING," MARCH, 1966.

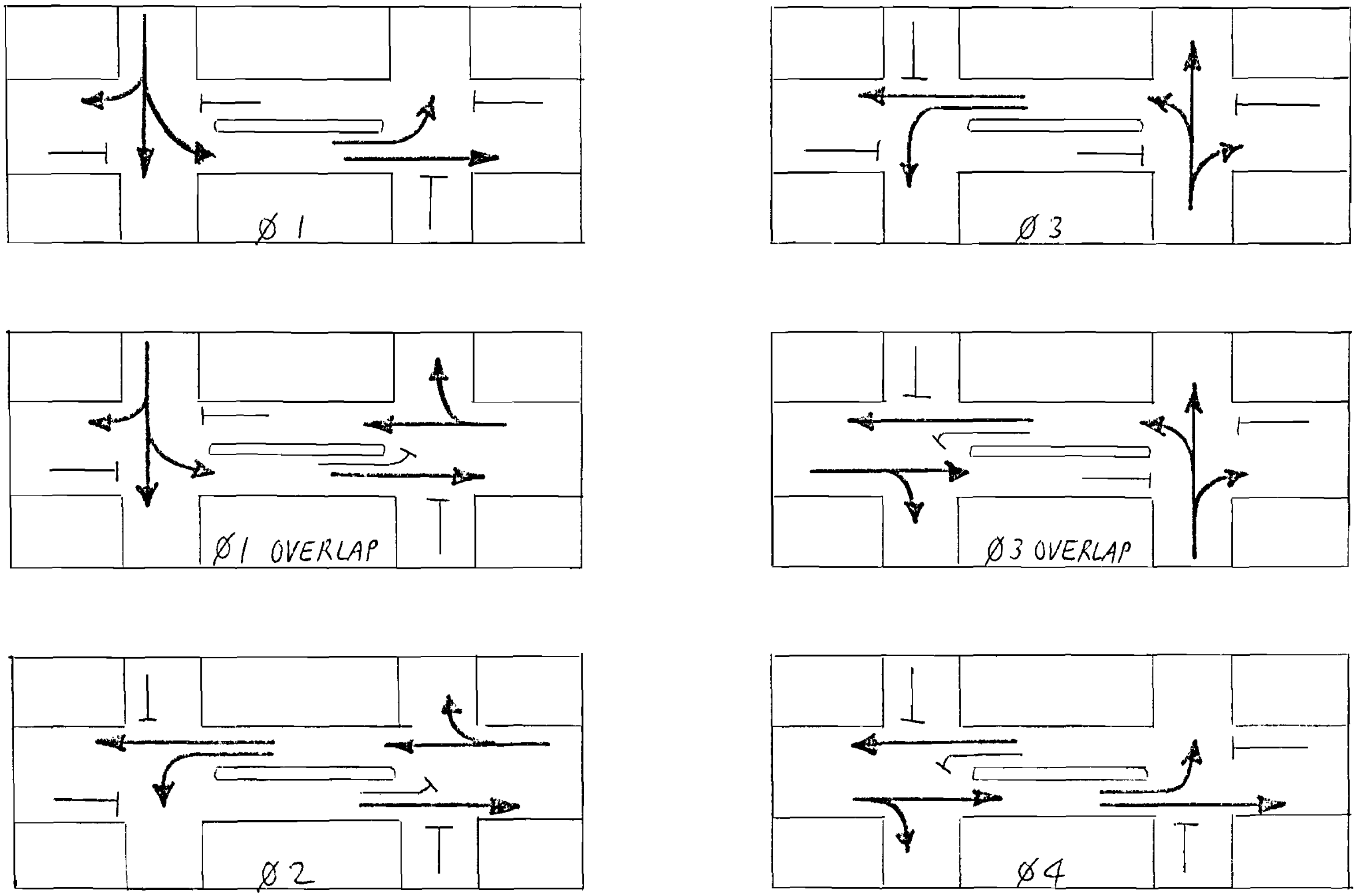


FIGURE 14: DIAMOND (FOUR PHASES WITH OVERLAPS)

This overlap utilizes the green time per cycle more efficiently and permits movement of large volumes through the interchange.

It could thus be concluded that a four-phase sequence with overlaps would be the best signal phasing for a conventional-type diamond interchange. The Recommendations section discusses an application at the I-20 ramps.

Analysis of Detector Devices

Presently, loop detectors are installed on all approaches for the I-20 diamond interchange, Wharton Drive, Great Southwest Parkway (T) and Cascade Road. Loops are in place for the side street only at Patton Drive, while at Gordon Road all approaches, except for the southbound Boulevard, are loop actuated.

In evaluation of the detection needs of a roadway with those traffic characteristics of Fulton Industrial Boulevard, various detector types require consideration.

Small-area detection and the dilemma zone problem

Conventional control at local intersections generally makes use of point detectors (eg. 6ft. x 6ft. loops) operating with locking detector memory circuits. Associated with the approach speed and the passage time from the detector to the stop bar is a so-called "dilemma zone." Figure 15 indicates a potential traffic situation which can create such a zone,without proper detection. A vehicle traveling at a constant speed, of say 45 mph, approaches an intersection and at the beginning of the yellow interval is some distance x from the stopline. A decision must be made whether to traverse the intersection or to stop instead. Within a certain range of distances, which vary with the approach speed, motorists will be uncertain

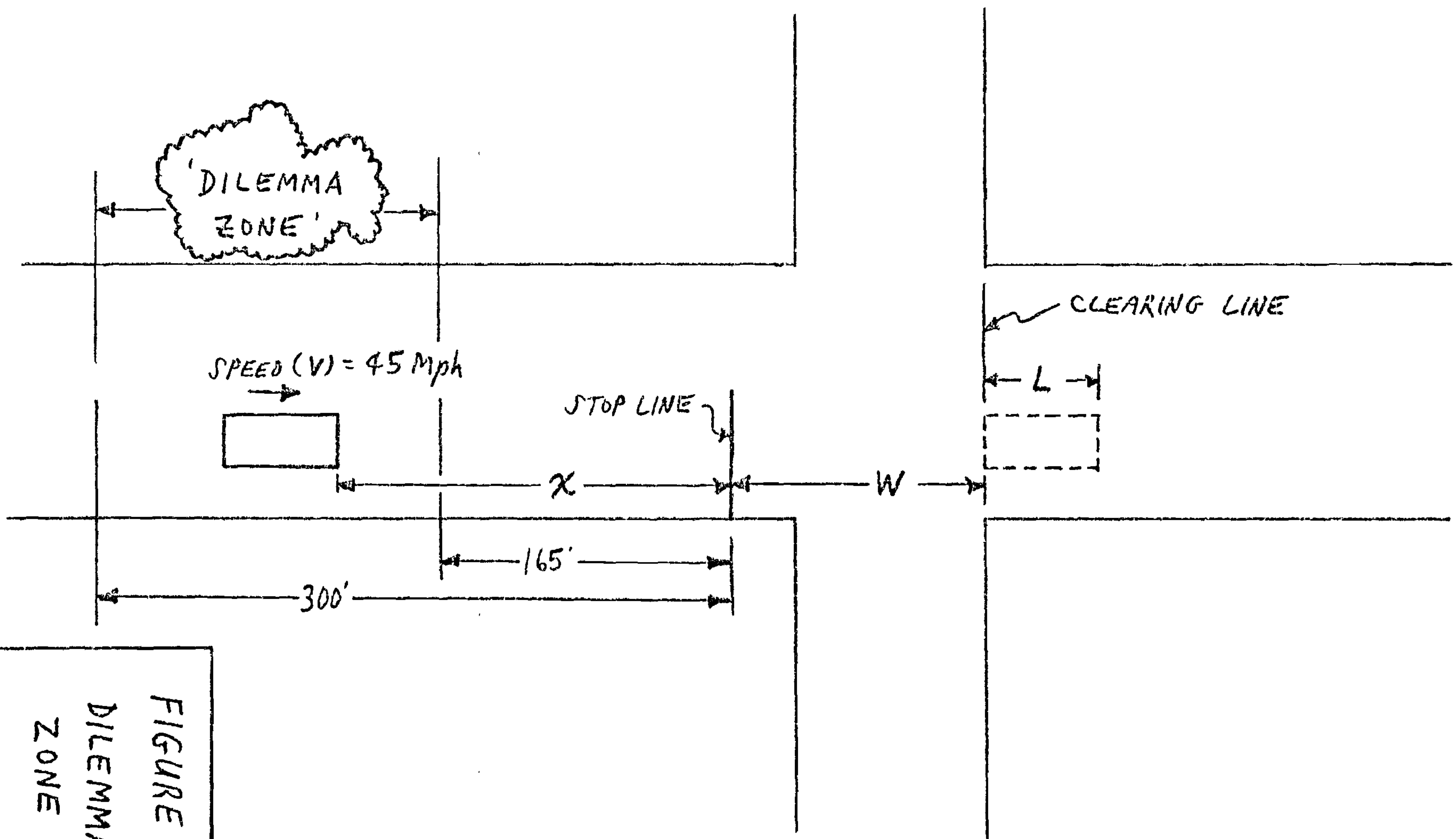


FIGURE 15
DILEMMA
ZONE

as to their chances of safely clearing the intersection. Safety hazards associated with resultant rear end or crossing movement collisions can be minimized by selection of a certain detector setback spacing from the stop line. It is necessary to detect a vehicle just before it enters this zone. Table 4 shows recommended distances based on a high probability of stopping (90 percent) for various approach speeds. At these spacing, upon detection, the controller will establish a new passage time for safe movement past the dilemma zone. If the yellow interval is reached just prior to the vehicle entering the correctly located small area detector, the driver will be free of indecision as to stopping. It should be noted that a "dilemma" was defined to occur within the probability of stopping limits of 10 to 90 percent (9). Additional information on the dilemma zone problem is contained in the February, 1974, edition of Traffic Engineering.

Nearly all detectors along the Boulevard are presently set too close to the stop lines for the traveled speeds. Further, the basic actuated controllers presently utilized do not have the internal capability to solve the dilemma zone problem. Detector logic thus needs to be added by an external device. Such external detection logic is available in the form of a "Green Extension System" (see the Recommendations section for specified locations). Extension of the green interval to carry a vehicle safely through the dilemma zone can be accomplished by means of a pair of small area detectors. Loop placement is a function of approach speeds and gaps (headways). Length of such loops is normally four feet and their width should be sufficient to cover all approach lanes. Each detector is connected to a timer which will hold (or stretch) the output for a determined period. A force-off timer is used on the loop furthest upstream. A gap must appear in both of the extension timer outputs before the controller will register a yellow change interval. The force-off timer on the upstream loop functions to deactivate the extension timer associated with that loop, if traffic

TABLE 4[10]

DETECTOR SPACINGS TO AVOID THE DILEMMA ZONE

Approach Speed (Mph)	Detector setback from stopline for 90% probability of stopping
30	175
40	250
45*	300*
50	350
55*	400*
60	450

*Interpolated

(10) Source: Traffic Engineering, February, 1974

does not gap put in the time set on the force-off timer.

Small-area detection and the ramp queue problem

The detection and discharge of excessive queues, such as those forming on the westbound off-ramp of I-20 during the morning peak period, can be handled by installation of a queue detector. The minimum length of such a loop must be such that it will bridge the open gap between a pair of waiting vehicles. The maximum length is a function of not allowing a continuous call at normal speeds. A length of 25 to 30 feet is generally used. The queue detector unit is a time delay device with external adjustments. For example, if the queue detector is set at 5 seconds there will be zero (no) output from the unit unless a vehicle remains in the loop greater or equal to the 5 second time period, otherwise it resets to zero. With semi-actuated controllers, the ramp becomes the actuated phase with the detector output calling for a maximum time greater than max 1 and thus extending the ramp green. With full-actuated controllers the queue detector would be used to disable the artery detectors. The queue detector is installed in addition to the phase C ramp detector at a location upstream which is fixed dependent upon ramp geometry and anticipated traffic volumes. This system relies on a basic actuated controller. The alternative approach to the queue discharge problem is to utilize a volume-density controller which would detect off-ramp traffic by the "cars waiting" external setting. Vehicle would be thus discharged by appropriate controller dial settings in advance of back-ups to I-20. A useful reference to small area detectors is found in Traffic Engineering, February, 1974 (11).

Large-area detection

These detectors use non-locking controllers and operate in the presense mode. The non-locking feature helps eliminate the problem of returning side street green to an empty cross street. Initial and vehicle interval settings can be substantially reduced to as low as zero and one second, respectively. A power head is an essential addition to long loop presense detection. The power head serves to detect small vehicles, such as motorcycles, and is most efficient when angled at thirty degrees to the stopbar. Figure 4 of the "Large-Area Detection At Intersection Approaches" report (12) should be referred to as an indication of proper alinement of the power head. This report, a copy of which has been furnished Fulton County, contains other specific commentary relative to large-area detection. Specific applications are noted in the Recommendations section.

The Traffic Adjusted Systems Concept

A traffic adjusted system selects offsets and cycle lengths from data furnished by sampling detectors, generally located near the extremities of the controlled section. Its components consist of sampling detectors, a master control assembly and local controllers. A local coordinating unit is a fourth element. A group of semi-actuated controllers are coordinated by a unit which receives inputs from the traffic adjusted master thorough the detectors. Parameters considered include volume and directional flow. A constraint is thus imposed that such a system not be used where a volume inversion is likely to be prevalent.

The master maintains a continuous comparison of inbound and outbound traffic volume levels. The heavier of the directions determines the background cycle length for the system. Offsets are determined by the difference between the directional volumes. System cycle lengths are developed based on the critical intersection capacity. Up to four different cycle lengths can be selected. Usually the shortest cycle is reserved for free operation which would ordinarily occur during the period between 7 p.m. and 6 a.m. Free operation is characterized by having the master disconnected from the local controllers. Thus, the locals function as isolated semi-actuated units during this period. If coordinated operation is necessary on a continuous basis, the two shortest cycle lengths may be made equivalent. When volume reaches its peak, the longest cycle length will be required. This is generally the case during morning and evening peak periods.

Determination of which cycle lengths to use during particular time periods is based on a study of average weekday fluctuations in the traffic volumes. Plots are usually diagrammed to cover the time period from 7 a.m. to 6 p.m. Fifteen minute increments are recorded directly from mechanical recorder counts. The series of figures 16 through 18 illustrate volume fluctuations along various segments of Fulton Industrial Boulevard. From these traces, particular peaks may be identified which require longer cycle lengths. It is important to avoid frequently changing offset shifts, since associated with each such step is a reduction in system efficiency.

Local controllers should be semi-actuated. In coordination these locals yield to cross street calls as regulated by that background cycle in effect. Background cycles are established by local coordinating units.

Local coordinating units may provide a maximum of three background cycles depending on the particular unit selected. Offsets are adjusted by external controls. Since the end of main street green is the only fixed point in a semi-actuated equipment situation, all offsets must be calculated on this basis. Splits used to calculate the required time space diagram are measured using local traffic conditions rather than considering the system as a whole. Coordinating units contain a permissive period which allows the local controller to yield to those side street calls placed just after the yield point. A maximum limit adjustment returns the controller to its main street phase in time to be given its predetermined split.

Sampling detector location is crucial, if the system is to properly

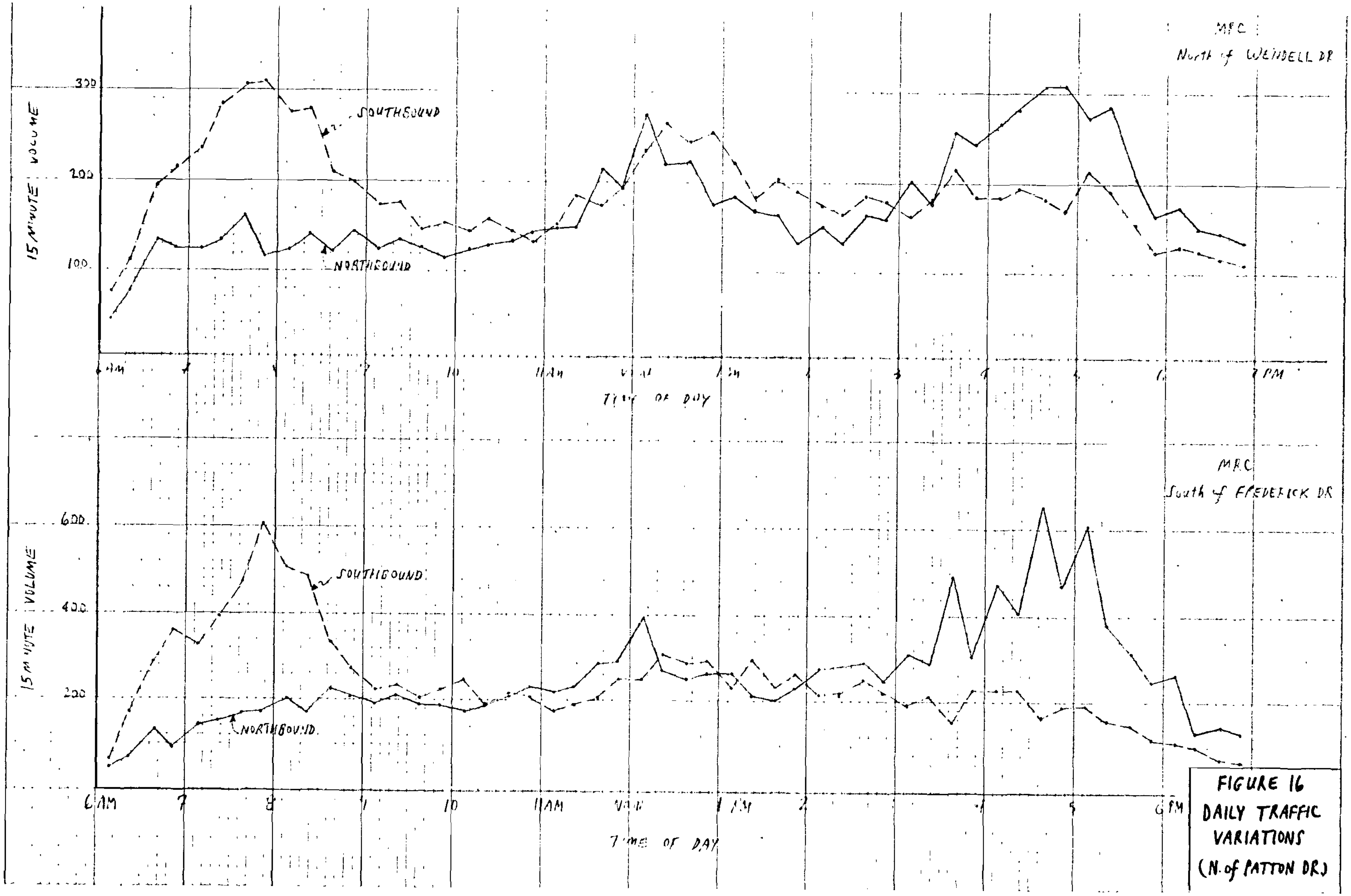


FIGURE 16
DAILY TRAFFIC
VARIATIONS
(N. of PATTON DR.)

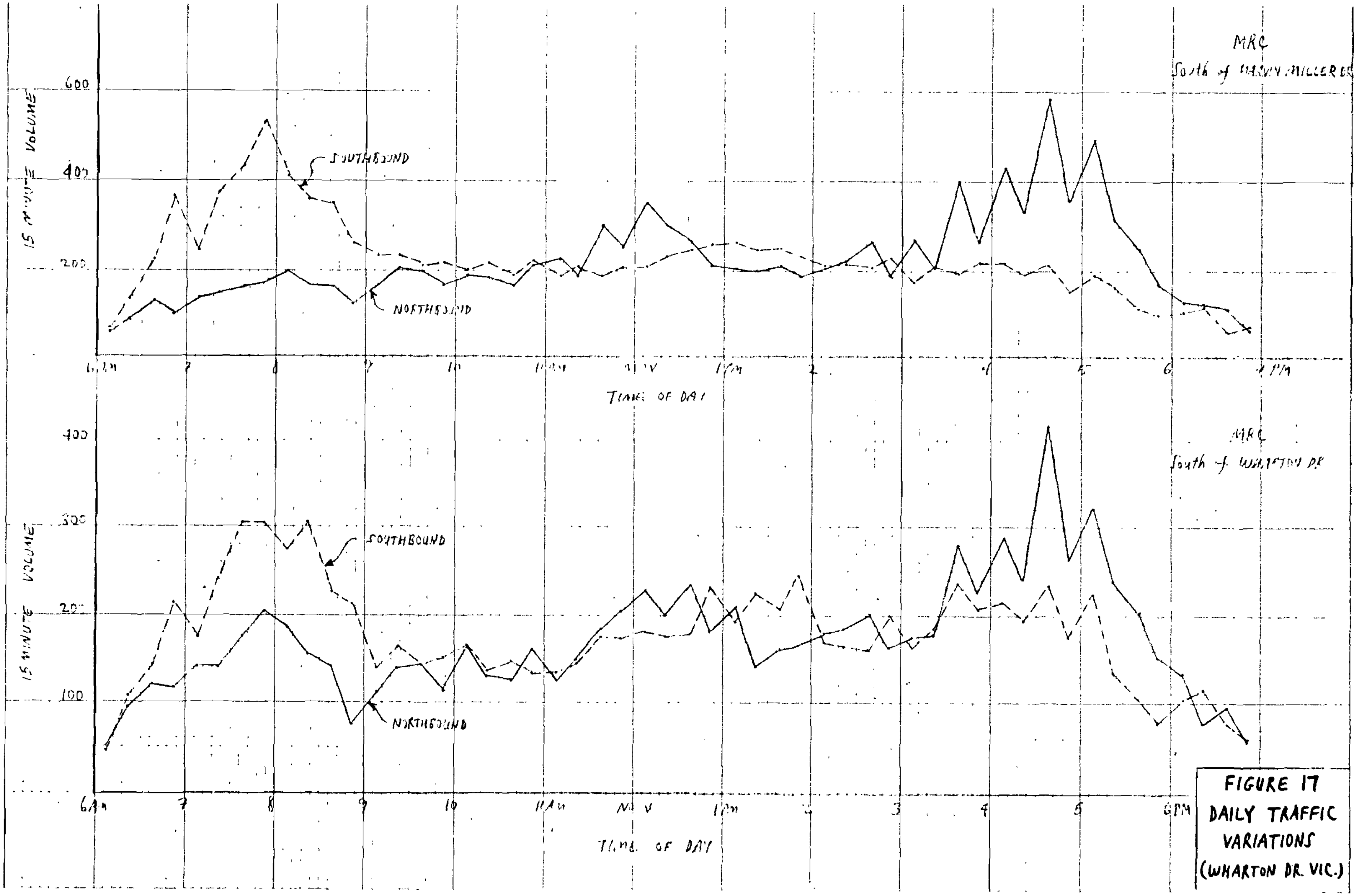


FIGURE 17
DAILY TRAFFIC VARIATIONS
(WHARTON DR. VIC.)

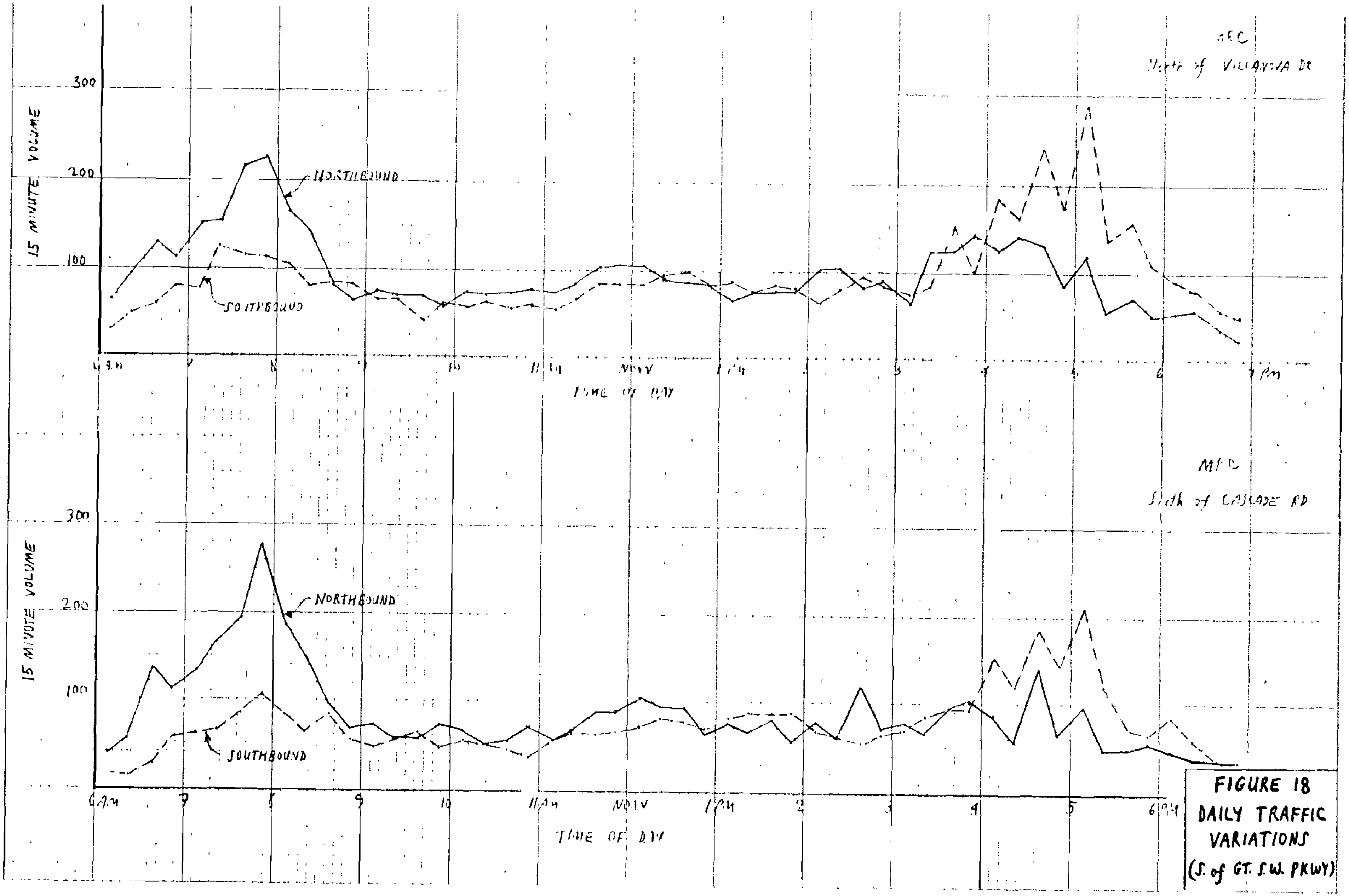


FIGURE 18
DAILY TRAFFIC VARIATIONS
(S. of GT. S.W. PKWY)

respond to actual traffic conditions. Usually loop detectors are used, on either approaches or single lanes which exhibit representative traffic patterns relative to the system. Detectors should be located at free-flowing sites.

A sync monitor unit is a desirable addition to a traffic control system having sync output. If a synchronization pulse fails to be transmitted from the master, the monitor allows local intersection controllers to continue to operate. When the master again transmits synchronization information, the system will automatically resume synchronous operation.

Analysis of Cycle Lengths

Studies of vehicle volumes, including the percentage of turning movements and trucks, yielded data on desirable splits of green time at isolated intersections. Mechanical recorder counts supplemented this information by providing relative proportionment of traffic during the peak hours. Speed data was developed from floating car runs utilizing a tachograph. Given these inputs, it was necessary to establish a preferred cycle length which would optimize progression between isolated intersections while maintaining adequate individual timing. SIGPROG, developed by Bleyl (13) was selected to determine such a cycle both during peak hour and average offpeak conditions. A FORTRAN program, SIGPROG, is combined with the TMSPAC plotter routine to plot an optimal time space diagram for a given data set. Initial runs were made to establish an optimum cycle (to the nearest five seconds) within a possible range of 50 to 120 seconds. A minimum time of 12 seconds was coded as desirable for cross street traffic where volumes did not warrant such an interval. Travel distances were scaled from State Department of Transportation plans or were measured in the field, as in the case of new cross street and driveway locations. Desirable progressive speeds were assigned based on field studies of actual traffic conditions under free flow. Review of SIGPROG results and of manual capacity calculations indicated that an offpeak cycle length of 50 seconds would be adequate to meet 1975 demands at a level of service "C". During the morning peak period a 50 second cycle, with provision for double cycling to 100 seconds at critical locations, would be required. These figures increased to 60 and 120 seconds during the evening peak period.

Analysis of Splits

The split is the ratio of time devoted to each phase at a signalized location. This phase split is determined on the basis of roadway characteristics and traffic volumes for critical approaches. In addition, no less than a minimum amount of time must be granted to the minor (cross) street movements. Table 5 indicates those splits determined for Fulton Industrial Boulevard between Frederick Drive and Boat Rock Boulevard (assuming signalization) based on current traffic counts.

All splits assume improvements advocated in the RECOMMENDATIONS section to have been carried out.

Access Guidelines - Analysis of Literature

The question of access versus capacity and safety is prevalent in attempting to provide adequate median openings while minimizing effects on level of service and likelihood of accident potential. Interruptions in traffic flow, such as those caused by vehicles turning off of or onto high speed roads, will produce various acceleration and deceleration maneuvers. A combination of these may require a sequence of responses or reactions which the average driver is not capable of successfully executing.

Median Opening Effects on Accident Rates and Level of Service

In order to correlate the relationship of median openings with significant variables, a study published in 1967 and conducted at North Carolina

TABLE 5 PHASE SPLITS

Location	AM Peak Period	Average Off Peak	PM Peak Period
Frederick Dr.	84%	68%	76%
Patton Dr.	81	68	71
Wharton-Mendel Drs.	68	68	62
Great S.W. Pkwy ("T")	68	68	58
Cascade Rd.	68	68	62
Boat Rock Blvd.	68	68	55

All splits assume improvements advocated in the RECOMMENDATIONS section to have been carried out.

State University, Raleigh, was reviewed (14).

The authors were, in fact, unsuccessful in their primary goals, i.e., to determine quantitatively the optimum median opening spacing on multilane divided highways. However, numerous useful results were noted, including correlation of accident-causing factors.

The data base was provided by over 6,000 separate accident records over 92 study sites during a 21-month period in 1963-64. Each accident was related by a distance measurement to a median opening. Data were divided by accident and location type, then were analyzed using multiple-regression techniques.

Since regression equations indicated that average daily traffic was the predictor of greatest magnitude (with respect to Student "t" values) and of most frequent occurrence, traffic volume was recognized as the most significant predictor. The frequency of median openings also ranked high in prediction significance.

Based on simple correlation coefficients a number of findings were made regarding both simultaneous and independent investigation of the variables utilized. Those found to be most useful for the Fulton Industrial Boulevard study include the following:

Simultaneous Consideration

- Roadside access increases as the number of median openings increases or the ADT volume increases.
- Rear-end collisions account for one-third of all accidents on four-lane, nonaccess-controlled highways.
- Unsignalized intersection and signalized intersections rank first and second as high-frequency locations for rear-end collisions.
- The number of rear-end collisions is less where storage lanes are provided.

Independent Consideration

- As volume increases, accidents at median openings with and without storage lanes increase.
- As access points increase, accidents of all types increase.
- As volume increases, accidents between openings increase.
- As access points increase, travel time increases.
- As access points increase, accidents at signalized and unsignalized intersections with storage increase.

Other findings of the study allowed some general conclusions to be made.

- As traffic volumes increase, usage of median openings rapidly becomes dangerous. When combined with intensive roadside development, usage of median openings under high-volume conditions becomes very unsafe.
- Signalization of median openings does not necessarily reduce the hazard of using openings under high-volume conditions, but tends to make the traffic flow more orderly by offering a more equitable distribution of time for each driver.
- As roadside development increases, and crossovers of any type are permitted, accidents will increase.
- Fewer accidents were found on sections with higher speed limits only because such speed limits were permitted in locations with low volume and low intensity of roadside development. Reduction in speed limit, when volumes are high and roadside development is intense, does not keep accident rates at a low level. The increased hazards associated with turning movements under high volume conditions far exceed the benefits derived by reducing the speed limit.

The study recommended that state highway departments give consideration to adopting a policy which would permit the predetermination of the specific location of all openings in the median on future construction of divided highways. Following such an approach, the approximate spacing and location of traffic signals would be preplanned to assure a travel speed and signal progression compatible with the desired level of service. This plan would also designate the location of all future openings and would prohibit future alteration of such locations and spacing.

Another recommendation was to encourage abutting facilities to design their development in accordance with efficient and safe use of the public highway. When a highway is altered to specifically serve abuttors, accident potential will increase and the level of service will deteriorate. Consequently, roadway function must be defined prior to the intensive development of roadside property so that, throughout its functional life, a divided highway will continue to serve the general public.

Finally, it was suggested that median opening spacing should be such that efficient two way progression results along the signal system. Signal spacing at median openings is dependent on average operating speed which reflects the roadway's level-of-service.

Roadway and Operational Characteristics Effects on Multilane Highway Accidents

A second study, also conducted in 1967, used the same data base as the preceeding investigation (15). Characteristics were defined as median width, speed limit, volume, level of service, access point frequency, intersection openings per mile, signalized openings per mile, and median openings per mile. All characteristics were then correlated with injury accidents. A multiple-regression analysis was performed so that effects on accident frequency of all site characteristics could be examined simultaneously.

Determination of effects of highway characteristics on the median opening accident rate was also considered. The Student's "t" value, which was calculated for each regression coefficient, indicated whether the variable corresponding to a coefficient had a significant effect on the accident rate being examined. The predominance of positive coefficients indicated that as the magnitude of the variable increased, the median-opening accident rate increased. When storage lanes were installed at openings, the median-opening accident rate was no longer significantly affected by the number of openings, excluding intersections, the median width, the speed limit, or the traffic volume.

Findings of this study showed that no one roadway characteristic was closely enough related to injury accidents to be identified as a primary cause. From a tabulation of regression coefficients for each independent variable corresponding to an accident type, it was noted that the roadway characteristics do influence the median opening accident rate. For almost all accident types, the accident rate tends to increase as the number of median openings, excluding intersections, increases. The two characteristics having the least effect on the accident rate were the median width and speed limit. As the width of the median increased, the accident rate, except for commercial vehicles, increased.

Other results were as follows:

- Injury accidents and total accidents are closely related and can be predicted from each other.
- The predominance of positive coefficients indicates that as the magnitude of the variable increases, the median-opening accident rate increases. This information tends to support a theory that the number of median openings of all types should be minimized.

AASHTO Access Guidelines

Both policy and standards have been developed by research sponsored by the American Association of State Highway and Transportation Officials (AASHTO) dealing with access controls for primary urban arterial streets (16).

AASHTO recommends that these policy guides be followed:

- Access control policies and standards should be related to the function classification of the facility.
- Each administrative agency should develop and adopt a master highway plan of facilities and indicate the functional classification of each together with appropriate standards.
- Policy and standards relating to intersections with other public streets should be based on intersection spacing criteria and not on the existence or location of cross streets.
- Guides for the location of direct access points on arterials should be developed on the basis of traffic operation on the arterial rather than on the basis of land ownership patterns.
- Curb cut permits should be issued jointly and should be based on an acceptable site development plan. Local and state traffic staffs should work directly with the developer in arriving at a mutually agreeable provision of access.
- Improved access control procedures need appropriate legislation to implement control standards.

ASSHTO also recommends the following guidelines for access control standards to result in minimum interference to through traffic:

- Intersection spacings of 1600 to 2000 feet between signalized locations is suggested.
- Direct access drives should be provided to major generators only under specific conditions.

- Separate right and left turn lanes should be provided at all intersections as appropriate to geometric conditions.
- Medians should have a minimum width of 15 feet at signalized locations and 30 feet at unsignalized locations where left turns onto the arterial are permitted. If left turns from only the arterial are permitted, the median width may be as narrow as 14 feet.
- Median openings that allow crossing traffic as well as left turns onto the arterial should be provided at public street intersections only.
- An exception may be exercised in the case of a major generator where the access point conforms in all respects to standards relative to spacing and design of intersections.
- All median openings should be of the "bullet-nose" design.

In dealing with spacing standards at signalized locations, more efficient and flexible operation can be achieved if intersections are uniformly spaced within a certain optimum range. In addition to providing a best progression plan under a particular condition set, spacing should also provide for system flexibility to adapt to fluctuations in traffic volume and to reduce road user costs.

The maximum spacing between signals is a function of the stability of platoons and the ability of drives to maintain appropriate speeds. In short segments, numerous situations occur which allow confirmation by the driver that his overall travel speed is or is not adequate for progression through a given system. Thus, for short signal spacing, platoons tend to be well defined. Studies (17) have indicated that 85 percent of vehicles remained in platoon after leaving an intersection 1/2 miles downstream. At a distance of a mile, 77 percent were still grouped in the platoon.

Certain considerations tend to further fix the upper and lower limits of a desirable spacing range. Spacing much larger than 2000 feet are generally not appropriate since, according to AASHTO;

- On an arterial having at-grade intersections traffic volumes are not likely to be high enough to produce further reduction in user costs.
- For a given number of turns, the longer the spacing, the larger the volume of turns that must be accomodated at the designated openings so that less green time may be available for through movement with any cycle length and multiphase operation.

AASHTO also investigated spacing as a function of road user cost and traffic volume (18). It was determined that for 4,000 vehicles per day per lane, an optimal spacing range of 1000 to 1500 feet occurred. As volumes inceased to 6000 and 8000 vehicle per lane per day, spacing also increased to 1800 and 2200 feet respectively. The study concluded that economic advantages could result at spacings up to 2400 feet at very high volumes.

Different sections along an arterial frequently have varying traffic volumes as is the case with Fulton Boulevard. As industrialization spreads southward volumes, as well as the location of relative volume sections, will increase and fluctuate respectively.

Median openings at unsignalized intersections on divided roadways should be provided only when there is sufficient room to screen the longest vehicle expected to use the opening. The length of the protected left turn slot must be sufficient to allow an acceptable deceleration and to store all turning vehicles. The storage length varies depending on the number of vehicles anticipated. Thus, median openings might be provided at spacings no less than the sum of the turn lane length (including the taper) plus an acceptable distance between the median opening and the beginning of the next downstream turn lane taper. Accepting no speed differential between through and turning traffic at the beginning of the taper and a deceleration rate of 6.5 feet/second/second, values denoted in Table 6 would result. A value of 25 feet per stored vehicle should be added for waiting queues. Accepting a 10 mph differential and a deceleration rate of 8.0 feet/second/second, values of Table 7 result. Under no circumstances should these absolute minimums be violated because of the uncomfortably high "g" forces involved over the 8.0 ft/sec^2 figure.

Stopping Probability

To gauge the improvement (if any) of the introduction of a progressive signal system, a series of six runs was made in each direction during the morning and evening peak periods using a standard vehicle. This data consitutes the "before" portion of any subsequent comparison of the probability of stopping at an intersection. Runs were conducted on July 23, 1975 under respresentative traffic conditions. Table 8 summarizes the study results.

Intersection Delay Time

At each of the intersections tested for the probability of stopping, an average delay per stopped vehicle was determined. Table 9 tabulates this data.

Accident Analysis

Over two hundred and fifty separate accidents were reported within the study area in 1974. Some locations had relatively few accidents, resulting in no serious injuries. Accordingly, it was attempted to establish criteria defining areas judged to be safety-deficient based on State Department of

TABLE 6

DESIRABLE MINIMUM DISTANCES BETWEEN MEDIAN OPENINGS

Arterial Speed (mph)	Spacing (ft) +25' per stored vehicle
40	530
45	670
50	780
55	910

TABLE 7

ABSOLUTE MINIMUM DISTANCES BETWEEN MEDIAN OPENINGS

Arterial Speed (mph)	Spacing (ft) +25' per stored vehicle
40	300
45	360
50	430
55	510

TABLE 8. PROBABILITY OF STOPPING

Intersection	AM Peak Period (%) NB	AM Peak Period (%) SB	PM Peak Period (%) NB	PM Peak Period (%) SB
Gordon Road	50	83	67	83
I-20 WB Off-ramp	50	83	33	67
I-20 EB Off-ramp	50	33	83	50
Patton Drive	17	33	67	33
Wharton-Mendel Drive	33	33	50	33
Great S.W. Pkwy ("T")	33	17	67	67
Cascade Road	67	33	50	50

TABLE 9. DELAY PER STOPPED VEHICLE

Intersection	AM Peak Period (%) NB	AM Peak Period (%) SB	PM Peak Period (%) NB	PM Peak Period (%) SB
Gordon Road	21	35	31	24
I-20 WB Off-ramp	27	44	27	101
I-20 EB Off-ramp	34	26	946*	15
Patton Drive	23	18	42	38
Wharton-Mendel Drive	18	15	24	20
Great S.W. Pkwy ("T")	16	19	22	19
Cascade Road	15	14	17	14

*Note: This approach was jammed on four of six runs.

Transportation data. This approach (using a State data base) proved unworkable since total accident rates were available only on a seven day week basis summmed over fifty-two weeks. All Boulevard counts represented weekday traffic and no conversion factor from five to seven day weeks was available. This was the case both for total accident rates and injury accident rates. Injury accident rates were available only in terms of a severity index. It was found that none of the Boulevard intersections, within the study area, appeared on the statewide list of high accident locations.

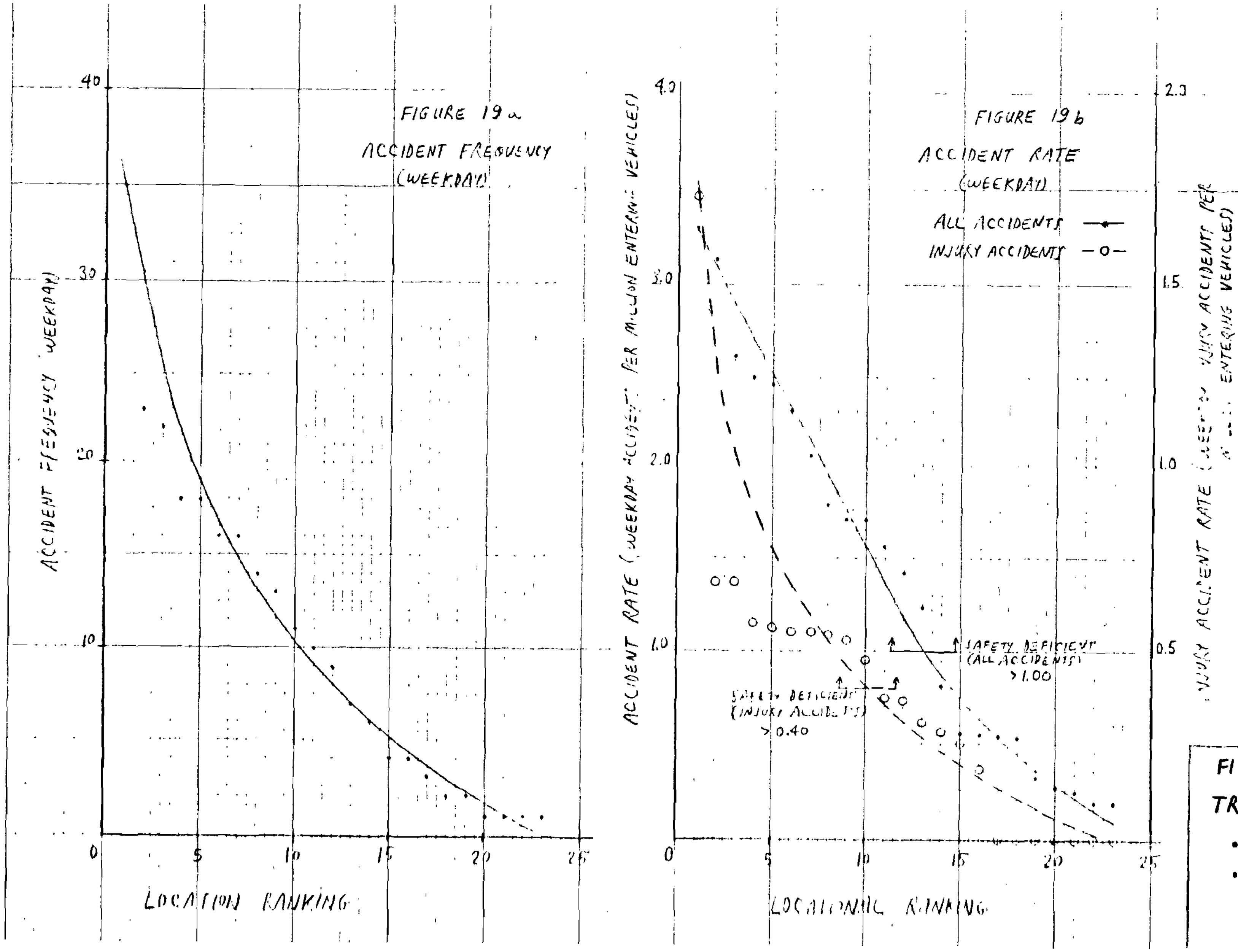
Accordingly, it was decided to plot accident data relative to frequency, total accident rates, and injury accident rates within the Boulevard study section. This would allow a relative comparison of the study intersections with one another. Figures 19a and 19b represent plotted data for the three aforementioned categories. This allowed the following criteria to be established to define sites which appeared safety-deficient with respect to other Boulevard intersections:

- Intersections with a <u>total accident rate</u> in excess of one accident per million entering vehicles.
- Intersections with an <u>injury accident rate</u> in excess of 0.40 accidents per million entering vehicles.

Frequency was not used as a criterion, as no breakpoint of that curve was apparent.

A study of accident patterns indicated that a typical collision occurred on a Friday in January between 4:00 PM and 5:00 PM. Table 10 represents a compendium of data for accident occurrence by time, day, and month.

Table 11 contains information used to plot the curves of figures 19a and 19b. This data includes accident frequency as a function of all accidents, weekday accidents, and weekday injury accidents. The average weekday traffic entering each listed intersection was determined from mechanical counts taken



FIGURES 19a & 19b

TRAFFIC ACCIDENTS

- **FREQUENCY**
- **RATES:**
  - **ALL ACCIDENTS**
  - **INJURY ACCIDENTS**

TABLE 10 ACCIDENT OCCURRENCE

DAY OF WEEK

Monday	40
Tuesday	51
Wednesday	47
Thursday	40
Friday	58
Saturday	13
Sunday	2
Unknown	1

MONTH OF YEAR

January	28	July	23
February	19	August	22
March	27	September	20
April	22	October	24
May	20	November	15
June	14	December	17
Unknown	1		

TIME OF DAY [1]

7PM	6AM	22	2PM	3PM	10
6AM	7AM	8	3PM	4PM	21
7AM	8AM	29	4PM	5PM	31
8AM	9AM	30	5PM	6PM	26
9AM	10AM	9	6PM	7PM	4
10AM	11AM	5	Unknown		2
11AM	NOON	12			
NOON	1PM	29			
1PM	2PM	14			

[1] Time periods extend from the hour through fifty-nine minutes thereafter. For example, 6 AM - 7 AM refers to 6:00 AM through 6:59 AM.

TABLE 11. ACCIDENT FREQUENCY AND RATES
(Jan. 1 - Dec. 31, 1974)

	No. of Accidents Total	Weekend	Injury	AWDT Entering Intersection	Accident Rate (1)	Injury (2) Accident Rate
ion Rd.	15	2	6 (2)	34,800	1.42	0.54
rchange Dr.	10	0	3	24,500	1,56	0.47
ell Dr.	18	0	2	29,800	2.31	0.25
) WB Ramp	38	3	3	40,700	3.30	0.28
) EB Ramp	26	4	6	40,800	2.06	0.56
:ley Dr.	18	2	0	36,000	1.71	0.00
:on Ind. Cr.	4	2	0	32,000	0.24	0.00
ierce Dr. Leg)	16	0	6	34,500	1.78	0.68
lerick Dr.	24	1	5	33,700	2.62	0.57
ierce Dr. Leg)	2	0	0	29,500	0.26	0.00
:on Dr.	14	0	3	31,500	1.70	0.36
in Miller Dr	4	0	0	28,500	0.54	0.00
inson Dr.	6	0	4	27,900	0.82	0.55
:ton-Mendel	18	0	4	28,100	2.46	0.55
dge Blvd.	3	0	0	21,800	0.53	0.00
ers Ferry (N. Leg)	7	0	3	21,700	1.23	0.52
ilip Lee Dr.	1	0	1	21,900	0.18	0.18
ig Dr.	1	0	0	21,500	0.18	0.00
ate Dr.	3	0	2	21,000	0.55	0.37
S.W. PKWY.	3	1	0	20,500	0.56	0.00
lanova Dr.	1	0	1	12,200	0.31	0.31
:ade Rd.	11	0	3	16,900	2.50	0.68
: Rock Rd.	9	0	5	11,000	3.13	1.74
	252	15	57			

Weekday accidents per million entering vehicles.

One injury accident occurred on a weekend at Gordon Road.

along the Boulevard and side street approaches. Accident and accident injury rates could then be directly calculated, the results of which also appear in this table.

Table 12 documents accident occurrence by type of collision, such as cross movement, rear end, etc. Special situations, such as wet pavement and non-daylight conditions, are also considered.

Personal injury resulted in 57 of the 252 accidents analyzed. Injury accidents tended to be scattered throughout the study area, with no emergence of predominant patterns relative to frequency. However, the injury accident rate at Boat Rock Road was nearly three times that of any other Boulevard intersection.

Using the established safety deficiency criteria, fourteen intersections and their proximate midblock sections (within 100 feet of the intersection) are discussed below. The series of 36" x 24" plans submitted may be used as a reference to specific collision diagrams. Table 13 serves as a key to interpreting the symbols used on the diagrams.

Boat Rock Boulevard

Sight distance is a particular problem for conflicts between southbound Fulton Boulevard traffic and vehicles exiting from the industrial park (EB). Trucks using the southbound right turn lane into Boat Rock Boulevard tend to screen southbound through-vehicles in the high speed lanes. Guidelines (21) indicate that a sight distance for stopping of 450 feet is desirable at speeds traveled by Fulton traffic. Yet, the present combination of horizontal curvature and truck traffic can reduce actual stopping sight distance to about 150 feet for a vehicle on the inside through-lane. Of nine accidents, five resulting in injuries occurred at this site in 1974. Additional traffic of considerable proportion is likely to be generated at this intersection as land further developes, compounding

TABLE 12. ACCIDENT STATISTICS

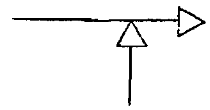
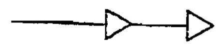
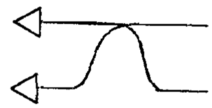
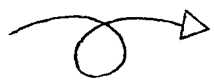
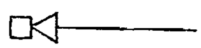
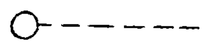
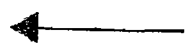
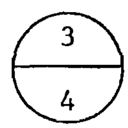
	ACCIDENT TYPE						SPECIAL CONDITIONS	
	CM	RE	SS	BK'ING (1)	F.O. (2)	OTHER	NON-DAYLIGHT ACCIDENTS	WET ROAD
Gordon Road	7	6	1	0	0	1	5	1
Interchange Dr.	7	2	0	1	0	0	4	1
Wendell Dr.	16	0	1	0	0	1	5	3
I-20 WB Ramp	11	20	1	1	1	4	5	4
I-20 EB Ramp	9	15	1	0	1	0	8	2
Shirley Dr.	9	5	3	1	0	0	4	4
Fulton Ind. Cr.	4	0	0	0	0	0	1	1
Commerce Dr. (N. Leg)	8	4	4	0	0	0	1	3
Frederick Dr.	16	3	2	2	1	0	3	2
Commerce Dr. (S. Leg)	0	1	0	1	0	0	0	0
Patton Dr.	3	10	0	1	0	0	2	0
Marvin Miller Dr.	2	1	1	0	0	0	1	1
Robinson Dr.	5	1	0	0	0	0	0	1
Wharton-Mendel	13	2	2	0	1	0	3	7
Aldredge Blvd.	2	0	0	0	1	0	0	3
Bakers Ferry Rd. (n. Leg)	2	3	1	0	1	0	0	0
Phillip Lee Dr.	0	0	0	0	0	1	0	1
Selig Dr.	1	0	0	0	0	0	0	0
Private Dr.	2	1	0	0	0	0	0	0
GT. S.W. Pkwy.	2	0	0	0	0	1	0	2
Villanova Dr.	1	0	0	0	0	0	0	0
Cascade Rd.	7	4	0	0	0	0	1	0
Boat Rock Rd.	7	1	1	0	0	0	1	3
	134	79	18	7	6	8	44	39

(1) i.e., Backing

(2) Includes Ped., Cyclist and Buses

TABLE 13

COLLISION DIAGRAM SYMBOLS

Cross Movement	
Rear End	
Sideswipe	
Rolled Over	
Backing	
Fixed Object	
Pedestrian	 (---Direction of Pedestrian Movement)
Injury Accident	
Injury Accidents Total Accidents	

the problem.

Cascade Road

While this location qualifies for consideration based on the aforementioned criteria, all data applies to the period prior to signalization. Of the eleven accidents occuring in 1974, seven involved crossing movements and the remainder were rear-end collisions.

Bakers Ferry Road (N. leg)

This site was the scene of seven 1974 accidents, three of which resulted in injuries. Located about 450' south of a vertical curve crest, sight distance is adequate for both the major and minor streets. Cross movement and rear end accidents have tended to predominate at this location.

Wharton-Mendel Drives

At least nine of eighteen reported accidents at this intersection appear to have resulted from either inadequate clearance interval timing, or because detector placement created a dilemma zone problem. Two other cross movement accidents occurred prior to signal installation (when control for the side streets was by stop sign). A pair of cross movement accidents could be attributable to vehicles exiting from the Anaconda driveway to make "U" turns around the northern island gore.

Robinson Drive

This location experienced six 1974 accidents, four of which resulted in injuries. One accident involved a rear end collision on the minor street approach, while all the others were cross movement accidents.

Patton Drive

Nine of fourteen accidents at this intersection involved rear end collisions on the high speed Boulevard. This indicates a dilemma zone problem. Other accidents resulted from crossing movements or, in one case, backing.

Frederick Drive

Most accidents at this site were associated with traffic exiting from Frederick Drive which were unable to clear the intersection. Such traffic has a sight distance of about 350 feet to the south due to a hill crest. When traffic is light, northbound vehicles tend to travel at speeds which do not allow adequate stopping sight distance. During peak period conditions, other cross movement accidents occur when through-street traffic in some lanes allows side street entries into the stream and others do not. The steep upgrade approach of the side street results in some backing accidents.

Commerce Drive (N. leg)

Eight of twenty accidents in this locale involved left turn traffic exiting from Commerce Drive striking or being struck by through-traffic in the travel lanes. Northbound Fulton Boulevard traffic also uses the shoulder as a through-lane, so that three through-lanes approach Commerce Drive from the south. Cross movement accidents result when northbound curb lane traffic stops and other lanes do not. Some left turners are either rear ended by other exiting vehicles in the median or are struck by southbound through-traffic beyond the median. Three rear end accidents occurred for northbound through-traffic approaching Commerce Drive. Collisions in which vehicles were cut-off or impacted occurred when center lane traffic made right turns into the paths of vehicles occupying curb lanes. Rear end collisions, often involving chains of vehicles, occur during the evening rush hour in the northbound lanes toward Shirley Drive.

Shirley Drive

Six of eighteen area accidents involved vehicles in the northbound center lane attempting to turn right into private drives between Commerce Drive and the Interstate 20 (EB) ramps. The remaining accidents involved predominantly rear end collisions during the evening peak period on the northerly approach to the signalized eastbound ramps.

I-20 Interchange

The rear end type accident is predominant in this vicinity accounting for thirty-five collisions. Another twenty accidents resulted from conflicting crossing movements. Shifting truck loads and fixed object collisions accounted for other safety problems. Right turns into private drives from the center lane continued to cause accidents.

Wendell Drive

Over half of the reported accidents near this intersection resulted from cross-movement collisions between left turn traffic from northbound Fulton Boulevard and southbound through-traffic. Private drives contributed to a pair of accidents.

Interchange Drive

Most accidents at this intersection could be traced to congested conditions resulting from northbound queues extending south from Gordon Road. Observations during the evening peak period indicate that traffic in the northbound left turn slot to westbound Gordon Road overflows into the high speed travel lane during almost every cycle. This overflow constricts through-traffic to a single lane and reduces efficiency of the Gordon Road intersection.

Gordon Road

Fifteen accidents were reported at this intersection during 1974, of which six resulted in injuries. Most collisions were of the cross movement and rear-end types. Turns into a private drive from the center lane, and also sidewipes, accounted for the remaining accidents.

SECTION III

-RECOMMENDATIONS-

RECOMMENDATIONS

This section advocates solutions to reduce congestion and accident experience along Fulton Industrial Boulevard. Immediate and future improvements are considered, the former having both generalized and specific objectives.

Short range proposals generally consist of the following: relocating and installing numerous detectors to avoid dilemma zone and false call problems, respectively; upgrading signing and pavement markings; providing bus turnouts off the existing shoulder at MARTA stops; and instituting an effective program for installing and maintaining signal and auxilary equipment.

Specific, short-range improvements should consist of the following: changing the signal phasing to allow left turns to Gordon Road on a permissive basis; an attempt to effectively coordinate the I-20 interchange using the existing three-phase controllers and coordination unit; installation of warning flashers at Shirley, Commerce (north leg) and Frederick Drives; and installation of intersection control beacons at Boat Rock Boulevard. Additional recommendations appear elsewhere in this Section.

Long range improvements should generally consist of the establishment of a progressive signal system extending from Frederick Drive to Boat Rock Road, inclusive. Some major geometric improvements will be necessary to maximize the system's efficiency as constrained by existing development. At the I-20 ramps a four-phase solution, similar to that proposed by Georgia Tech in 1973 for the Roswell Road interchange with I-285, may be required if the existing signal equipment proves ineffective in meeting future traffic demands.

Immediate Recommendations

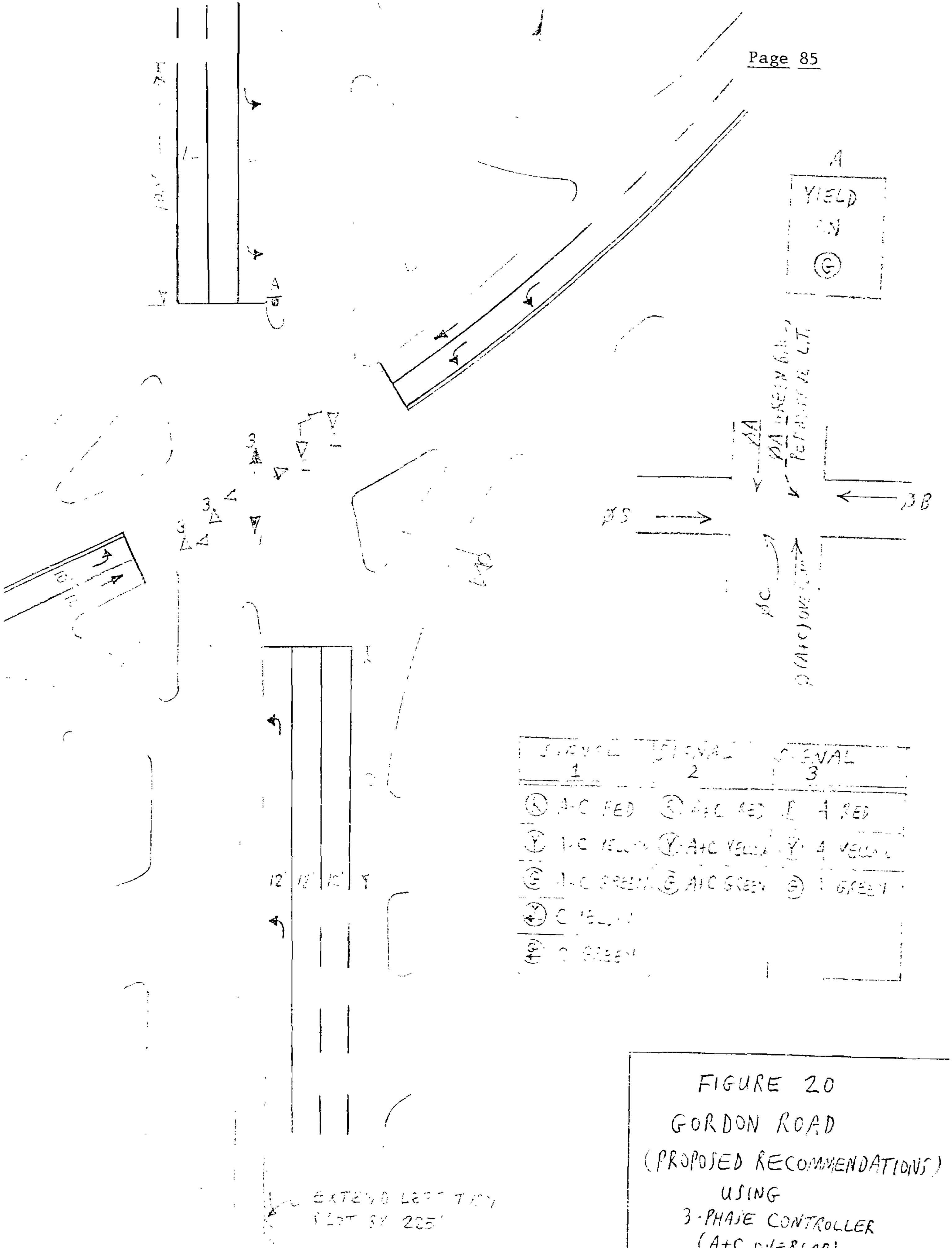
The Analyses section indicates that presently there are capacity and safety problems along the Boulevard capable of early correction. Some of these are general in nature while others are more specific. The following paragraphs deal with both cases:

Specific Considerations

Short range recommendations unique to a particular intersection are covered in this sub-section over the entire study area, beginning with the northernmost intersection and proceeding south.

Heavy left-turning movements from the northbound Boulevard to westbound Gordon Road are the primary problem at this intersection. The present left-turn display needs to be changed from its present exclusive indication to a permissive display, in order to minimize overflow of the left-turn bay. Due to relatively low speeds along this section of the Boulevard and the lack of sight distance problems this solution is feasible. The use of three phases, including an overlap, will permit the left turn bay to clear, thus reducing rear end accident potential between vehicles overflowing the bay and those in the higher speed lanes.

The existing ASD Model 1826N(M2) controller can be retained since it is a three phase unit capable of handling the proposed sequencing. Both minor movement controllers should be removed from the cabinet. Figure 20 illustrates the plan view of this intersection and denotes the displays relative to particular approaches. The generalized output of a three phase controller with overlap A + C appears in table 14. Its application to Gordon Road is shown in table 15. When the left-turn phase (C) ends, the equipment gives a yellow arrow rather than a circular yellow on signal face 1 (the heavy left turn maneuver). The yellow arrow is appropriate for terminating leading



SIGNAL 1	SIGNAL 2	SIGNAL 3
R A-C RED	R A+C RED	R A RED
Y A-C YELLOW	Y A+C YELLOW	Y A YELLOW
G A-C GREEN	G A+C GREEN	G A GREEN
C YELLOW		
C GREEN		

FIGURE 20
GORDON ROAD
(PROPOSED RECOMMENDATIONS)
USING
3-PHASE CONTROLLER
(A+C OVERLAP)

SCALE 1"=50'

TABLE 14

GENERALIZED OUTPUT OF A 3-PHASE CONTROLLER
(WITH OVERLAP A + C)

	Phase A		Phase B		Phase C	
	Interval					
Phase	1	2	3	4	5	6
A	G	Y	R	R	R	R
B	R	R	G	Y	R	R
C	R	R	R	R	G	Y
(overlap A + C)	G	Y[a]	R	R	G	G[b]

[a]Indication is green if phase C is next.
[b]Indication is yellow is phase B is next.

TABLE 15

3-PHASE CONTROLLER WITH OVERLAP
(GORDON ROAD)

	Phase A		Phase B		Phase C	
	Interval					
Phase	1	2	3	4	5	6
A	G	Y	R	R	R	R
B	R	R	G	Y	R	R
C	Phase C red is not wired				G	Y
(overlap A + C)	G	Y[a]	R	R	G	G[b]

[a]Indication is green is phase C is next
[b]Indication is yellow if phase B is next.

left turns. With lagging left turns, however, it is preferable to give the circular yellow to all faces on the approach because all lanes on that approach are to be stopped. However, if complete skipability is to be maintained there is no way to produce this yellow display.

The opposing (SB) left turn slot handles very light traffic. This movement should operate on a permissive basis during the phase A green. A special sign (see figure 20) should be erected for this bay.

A long loop detector, 40 to 50 feet in length and operating in the presense mode, should be added to the aforementioned critical left turn lane to maximize its operating efficiency. The use of "delayed-call" detection will prove most beneficial during off-peak periods when the controller is dwelling in phase A (see figure 20). At this time, a vehicle turning left onto Gordon Road would not require phase C to be called unless the vehicle were forced to wait in the turn bay longer than a predetermined internal, due to opposing traffic.

Small area detectors serving Gordon Road should be relocated.

Auxilary equipment, particularly the eight inch lens sections, should be upgraded. Twelve inch sections should be a standard item. Modifications of the displays to conform to those shown in table 15 and figure 20 need to be undertaken.

Due to intersection geometrics, the left lanes of both Gordon Road approaches should be clearly marked and signed for left turns only. The curb lanes should be identified for through (straight) maneuvers only. Figure 20 indicate recommended pavement markings.

The I-20 interchange has been responsible for creating the major bottleneck along the Boulevard. The need is to use less artery green time, in a more efficient way. The immediate solution would be to insure that the CR-16 is doing its job and that the pair of 1826N(M2) controllers is functioning correctly. If our studies at the interchange indicate that this equipment is not capable of handling the traffic demand, then equipment of the type discussed under Future Recommendations will likely be advocated by Georgia Tech. Figure 21 illustrates immediate signalization and geometric improvements suggested in the vicinity of the I-20 interchange. Phasing would follow that shown in figures 6 and 7 of the INVENTORY section.

Signal head displays need to be changed from circular indications to arrows as shown in figure 21 according to accident analyses. These displays should also be optically programmed. At the northern intersection, several collisions have resulted from sideswipes occuring on the double left turn maneuver. Others, involving ramp traffic and artery left turns, can be attributed to driver confusion as to which signal head(s) are controlling their approach. The geometrics of this interchange are complex to the degree that "DO NOT ENTER" type signing needs to be complemented by directional signal displays. Arrow indications should be added as an additional signal head for right turns from the Boulevard to both I-20 ramps, since drivers become trapped occasionally in the right turn lanes. These right turn displays would complement existing overhead signing. A third new head should be added to control the northbound left turn slot which is to be extended south to Shirley Drive.

New long-loop presence detectors should be located in coordination with the lengthening of the left turn bays (recommended below). The ramps, also, would benefit by long-loop presence detection. All long loops should include "powerheads," to detect motorcycles.

The small-area loops on the artery are too close to the stop bars; they should be replaced by new loops located as shown in figure 21.

Detector disconnect relays need to be added at both cabinets to control loop detection of the internal through-movements. The AO detector, for example, allows extension of phase A (see figure 7) until there is a phase B actuation, at which time it should be disconnected. During phase B, exten-

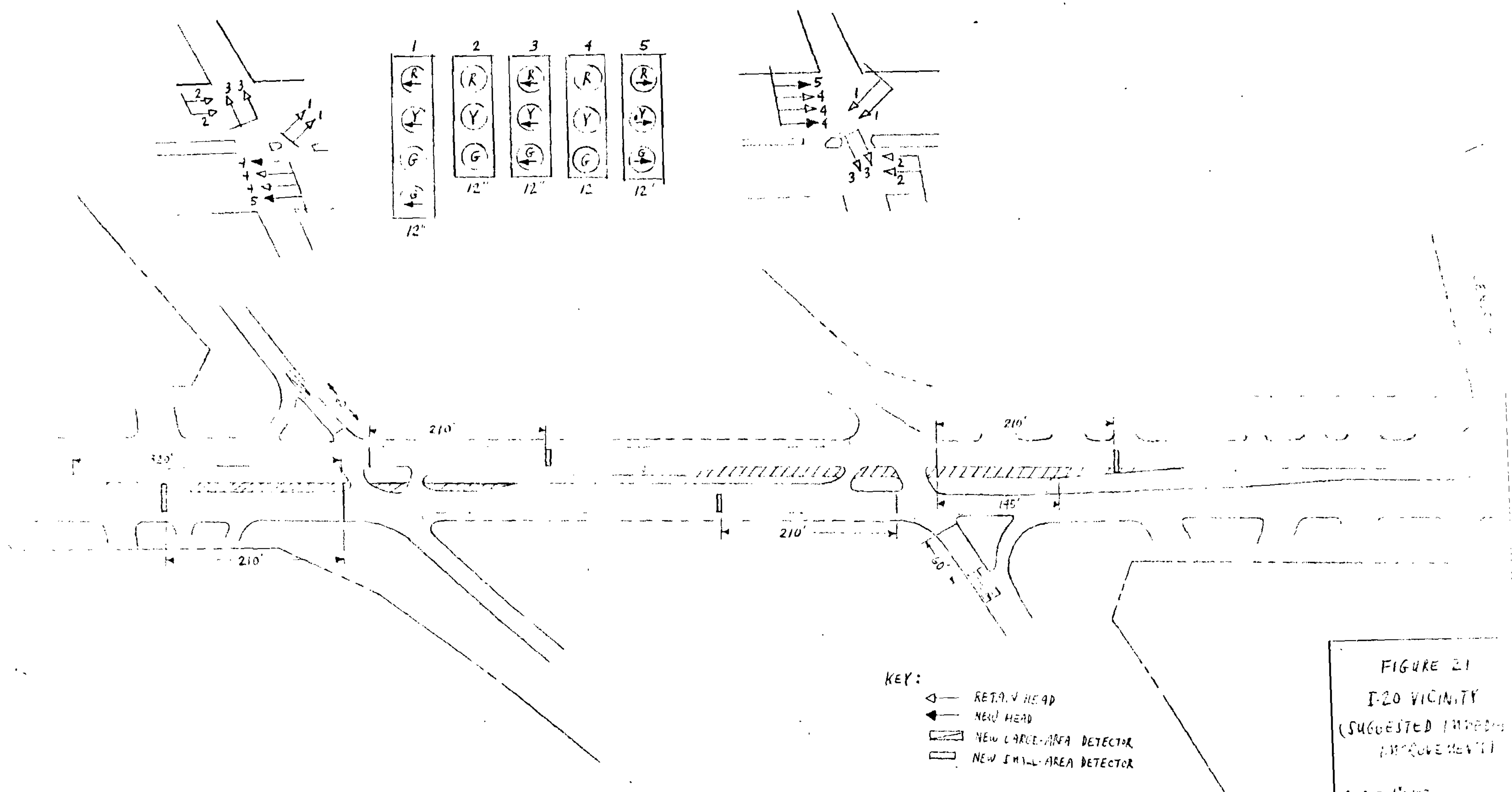


FIGURE 21
I-20 VICINITY
(SUGGESTED IMPROVEMENTS)

SCALE 1"=100'

sions will be given if a call is also present on phase C (the ramp). Otherwise, the disconnection will continue (ie. AO detector will remain disconnected). During the timing of the ramp, the disconnected detector will be reconnected to call phase A, if there is no phase B call. Otherwise it will remain disconnected. The DO detector functions similarly.

Geometric improvements are necessary at the interchange to accomodate present as well as future demands. There is an immediate need to lengthen the left turn bays for both the northbound and southbound Boulevard; the additional storage is required to avoid blocking of the through-lanes by vehicles overflowing the left-turn bays. Figure 21 indicates distances which should suffice to meet 1980 volumes. Associated with these extensions will be the need to revise signing and pavement markings.

Shirley Drive is presently being used as a test site for "do not block intersection" signing in order to create gaps for left turn traffic exiting Shirley Drive and desiring to travel southbound on the Boulevard. The success of the installation is, in part, dependent on gaining the driver's attention. It is also necessary to command the driver's respect, in part with the aid of police enforcement. Wig-wag flashers should be added to the sign at this location, to draw attention to the signing.

Commerce Drive (N. leg) is a location whose egress problems during the evening peak period are similar to those of Shirley Drive. It is suggested that well-located and attention-commanding "do not block intersection" signing also be installed at this location, along with flashing indications.

Frederick Drive also has an evening peak hour problem in that vehicles desiring to enter northbound Boulevard traffic have difficulty in finding gaps. In offpeak periods sight distance to the south is restricted for this traffic. Several serious accidents on this account occurred at this intersection in 1974. Signing and integral flashers need to be added at this location similar to those recommended at Shirley and Commerce Drives.

Patton Drive is presently experiencing an accident problem attributable to the "dilemma zone" problem found on high-speed signalized approaches. Driver confusion is resulting in numerous rear end collisions. A green extension system needs to be added to both Boulevard approaches to eliminate this accident hazard. Such a system attempts to solve the dilemma zone problem by detecting a vehicle just before it enters this zone. Upon detection, the system extends the green time sufficiently to give the vehicle safe passage through the zone. The green extension system concept is discussed in greater detail in the ANALYSES section (see Small-area detection and the dilemma zone problem).

A second problem, particularly during off-peak periods, results from minor street vehicles clearing the intersection and leaving a call to an empty street. This situation is especially prevalent since the Right Turn On Red law has become effective. Any unneccessary change of signal indication creates an extra opportunity for rear end collisions along the Boulevard, which experiences high speeds at this intersection. Therefore a delayed call detector using a 40 foot by 5 foot loop should be installed at the stop bar. The existing detector should be disconnected. This delayed call system need only be added at the eastbound approach until such time as Patton Drive is extended to the east and traffic increases on that westbound approach.

Wharton-Mendel Drives should receive the same treatment as Patton Drive to remedy problems caused by presense of dilemma zones on the Boulevard. In addition to adding a green extension system, long loops should be added to each approach lane of both minor streets due to the aforementioned problem of calling an empty street. Long loops should extend 40 feet behind the stop bar for both Wharton and Mendel Drives. Pavement markings need to be added to Wharton Drive. A pair of 12 foot approach lanes should be designated with the left lane being marked for left turns only. The right lane should be marked for right turns and the through (straight) movement.

Great Southwest Parkway is another intersection located along a high speed section of the Boulevard. Although 1974 accident data only revealed a single collision which would qualify as resulting from a dilemma zone problem, this location should be regarded as a potential candidate for a green extension system. It is suggested that the County review accident data at this location on an annual basis, commencing at the time all twelve months of 1975 records become available, and install a green-extension system at such time as dilemma-zone-attributable accidents are found to significantly increase.

Long presence loops with delayed-call capability should be installed 40 feet behind the stop bar for the Parkway approach. Double left turns should be permitted from the stem of the T into Fulton Industrial Boulevard. Pavement markings in the curb approach lane should designate the option of a right or left turn. Signing should complement the markings.

Cascade Road presents a situation in which accident records for 1974 do not reflect the present signalized conditions (signals were installed in the spring of 1975).

Accident records should be checked, this time commencing when the first twelve months of data are available, to determine the need for a green extension system. High speed traffic and the present detector locations indicate that the dilemma zone will be a problem. Long loops, extending 40 feet behind the stop bar should be added to both the Cascade Road and Great Southwest Parkway (south leg) approaches. The Parkway approach should be marked for right turns only from the curb lane. Left turns and straight movements should be permitted from the left approach lane.

Boat Rock Road presently experiences the highest injury accident rate, and the second highest accident rate for all collisions, along the study section of the Boulevard. The location is primarily dangerous because it is sited near the middle of a three degree curve. Southbound trucks intending to turn west into Boat Rock Boulevard screen other southbound high speed traffic from the sight of drivers waiting on the Boat Rock Boulevard approach. Three injury accidents resulted from this particular cross movement conflict in 1974. It is recommended that a pair of intersection control beacons be span wire mounted to regulate this intersection. A flashing yellow display would be directed toward artery traffic and a flashing red indication to the minor street approaches. The erection of these beacons will warrant installation of "signal ahead" signing for each Fulton Industrial Boulevard approach. This signing should be of the "W3-3" variety, the same as that used at the Great Southwest Parkway T intersection.

General Considerations

Additional short range recommendations can be applied throughout the study area. Topics noted below include: improvements to signing and pavement markings; the construction of bus turnouts; and the need for implementing an installation and maintenance program to increase equipment efficiency and reduce citizen complaints.

The following signing needs to be rectified throughout the study area:

a) Signing street names with "residential type" indications results in drivers slowing abruptly in an attempt to read the legend. The lack of identification by abutting land uses adds to driver confusion. A new standard should be developed which will incorporate clear messages and proper placement, particularly to benefit transitory operators (ie. truck drivers) unfamiliar with the area.

b) Signs of all varieties should be upgraded to conform with recommended (22) standards. The SIGN INVENTORY, of Appendix E, functions to identify non-conforming or substandard indications.

Pavement markings tend to be well worn by heavy traffic in most locations. It is advised that plastic markings be adopted at all signalized intersections and that a painting program, if it exists, be modified to include important facilities, such as the Boulevard, on a more frequent basis. Again, standards (23) should be adhered to in all applications.

Bus turnouts should be provided adjacent to stops to permit MARTA vehicles to safely stand while loading and unloading passengers. Initially, these turnouts can consist of paved areas. In curbed locations, curbing should be relocated to provide necessary storage. At least one accident involving a bus protruding into the curbside travel lane occurred in 1974.

Proper installation and maintenance are key factors in assuring that expected performance will be realized. Particular problems have been noted in detector installations, lightning protection, wiring and controller dial settings.

On August 22, 1975, a meeting at Georgia Tech on the subject of loop detectors was attended by the Fulton County Traffic Engineer, the two Fulton County Electricians, and Dr. Parsonson and Mr. Stupar of Georgia Tech. Dr. Parsonson reviewed with the group the difference in characteristics between the crystal-type detector unit and the modern, Sarastoa-type unit that is capable of tracking environmental drift. Emphasis was placed on recommending that the few remaining crystal-type detector units in the Fulton County inventory be used only at non-critical locations. It was stressed that they should never be used at the interchange of I-20 West and Fulton Industrial Boulevard, a locations at which top signal efficiency is a must.

The Fulton County electricians stated in this meeting that many loop detectors in the County are inoperative because the loop lead-in in the pavement has been shorted to ground by the pounding action of heavy traffic.

Dr. Parsonson then led a discussion of loop-detector-installation materials and procedures that have proven satisfactory in the experience of the Bureau of Traffic Engineering of the City of Atlanta. A key recommendation by Dr. Parsonson was to blow out the sawed slot thoroughly with compressed air and to seal in the loop wire with a gun-grade asphaltic caulking compound.

Lightning protection should be added to equipment by referring to the technical memorandum on "Lightning Protection for Traffic Signals, July 18, 1974" previously sent to the County. An eight foot, hard-driven ground rod, using well bonded #6 or #8 size wire, is recommended. The practice of using only a power company neutral as a ground should be avoided.

Wiring should be inspected and upgraded where required, particularly at the Wharton Drive controller cabinet. Undersize wiring to a ground rod should be replaced by #6 or #8 size wires at the I-20 controllers and other locations where required.

Controller dial settings should only be adjusted by, or upon specific instruction from, the County Traffic Engineer. This procedure will result in timing problems being brought to the direct attention of the Traffic Engineer and in proper settings being established based on study of traffic condtions.

Equipment should be periodically, at least weekly, checked on an operational basis. This degree of inspection is warranted by the Boulevard's significance as a traffic carrying artery.

Recommendations for Future Improvements

This section deals with solutions which should be implemented in the years to come to ensure continued provision of adequate capacity and reduction in potential accident occurrence along the Boulevard.

To maximize the effectiveness of these future suggested improvements, short range recommendations covered in the preceding sub-section should have been met, particularly those noted under "Specific Considerations."

"Long range improvements" is herein intended to refer to a time period commencing about 1980. At that time, projected traffic data indicates that the need for a coordinated signal system will be likely. It is around this concept of providing a progressive signal system that most of the "auxiliary" improvements south of, and including, Frederick Drive are advocated. Such auxiliary improvements would include geometric alterations and the provision of new access and frontage roads. North of Frederick Drive, a different signal strategy is proposed in which the I-20 diamond would continue to function on an internally coordinated basis, though isolated from any Boulevard progression system. Gordon Road will remain isolated unless tied into a system to the north, east or west. Changes to existing circulation patterns will be required between Frederick Drive and the Southern I-20 intersection and in the Robinson Drive vicinity.

Proposed Traffic Adjusted System

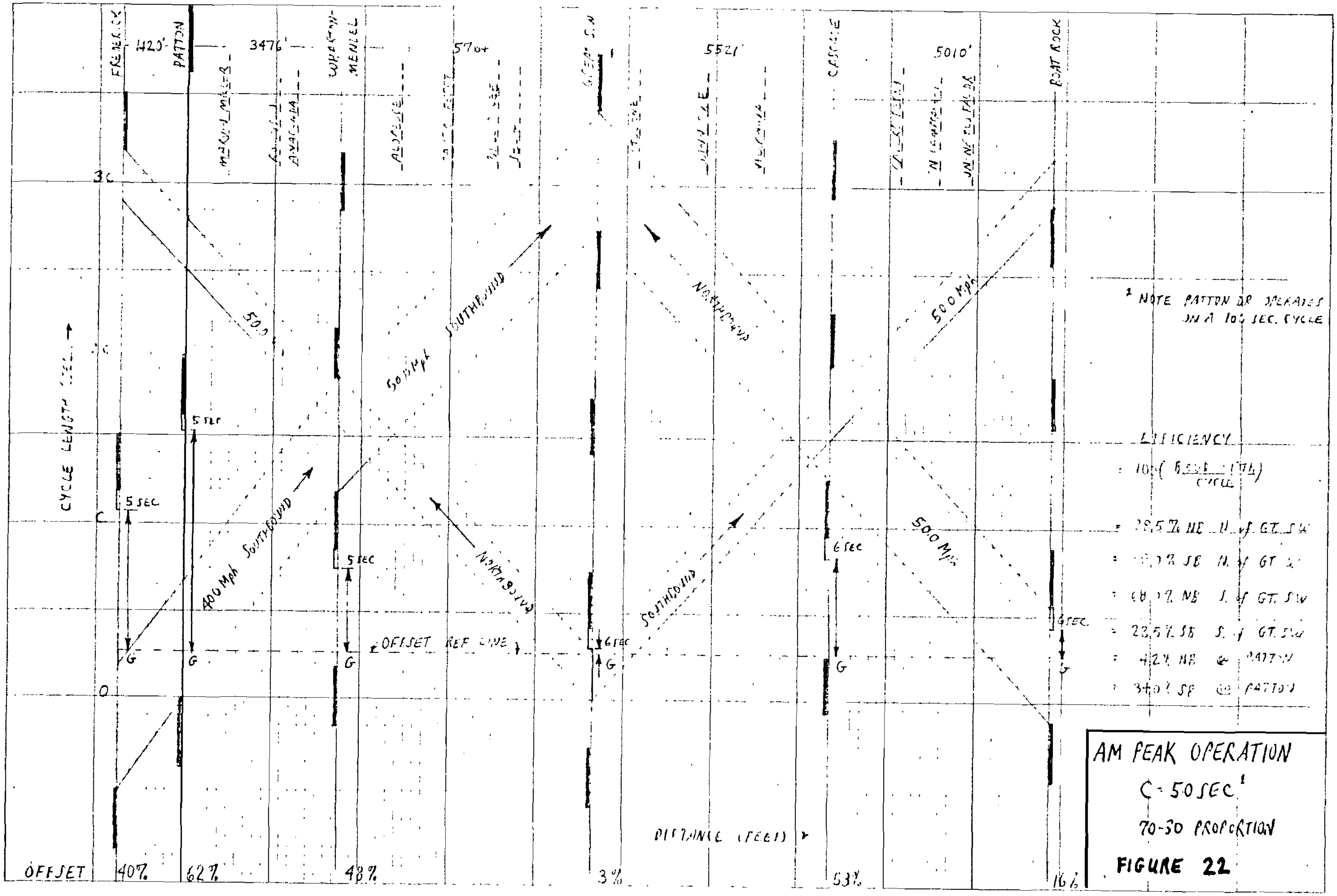
It is advocated that the segment of this study area between and including the intersections of Frederick Drive and Boat Rock Road be eventually included in a progressive signal system to meet projected traffic demands. Presently, no additional signals are warranted by traffic volume; however, based on past growth trends, Boat Rock Boulevard and the Frederick Drive - Commerce Drive (north leg) area will be the next locations to qualify for signalization. The "Accident Experience" warrant of the Manual on Uniform Traffic Control Devices (24) will be satisfied at these locations. With traffic growth of over 50 percent projected within the next five years, other intermediate locations will eventually qualify for signalization.

The primary consideration in locating additional isolated signals will be the meeting of MUTCD warrants and the access standards, as discussed in the ANALYSES section. Further, in meeting the MUTCD "Systems Warrant", or other warrants, intermediate signals need to be located so as to result in a desirable overall level of efficiency as determined by time-space diagrams. Accordingly, figures 22 through 24 were developed as the basis for a efficient progressive system. Achievement of the band widths shown is dependent upon those geometric improvements being completed which provide necessary approach widths at signal system locations.

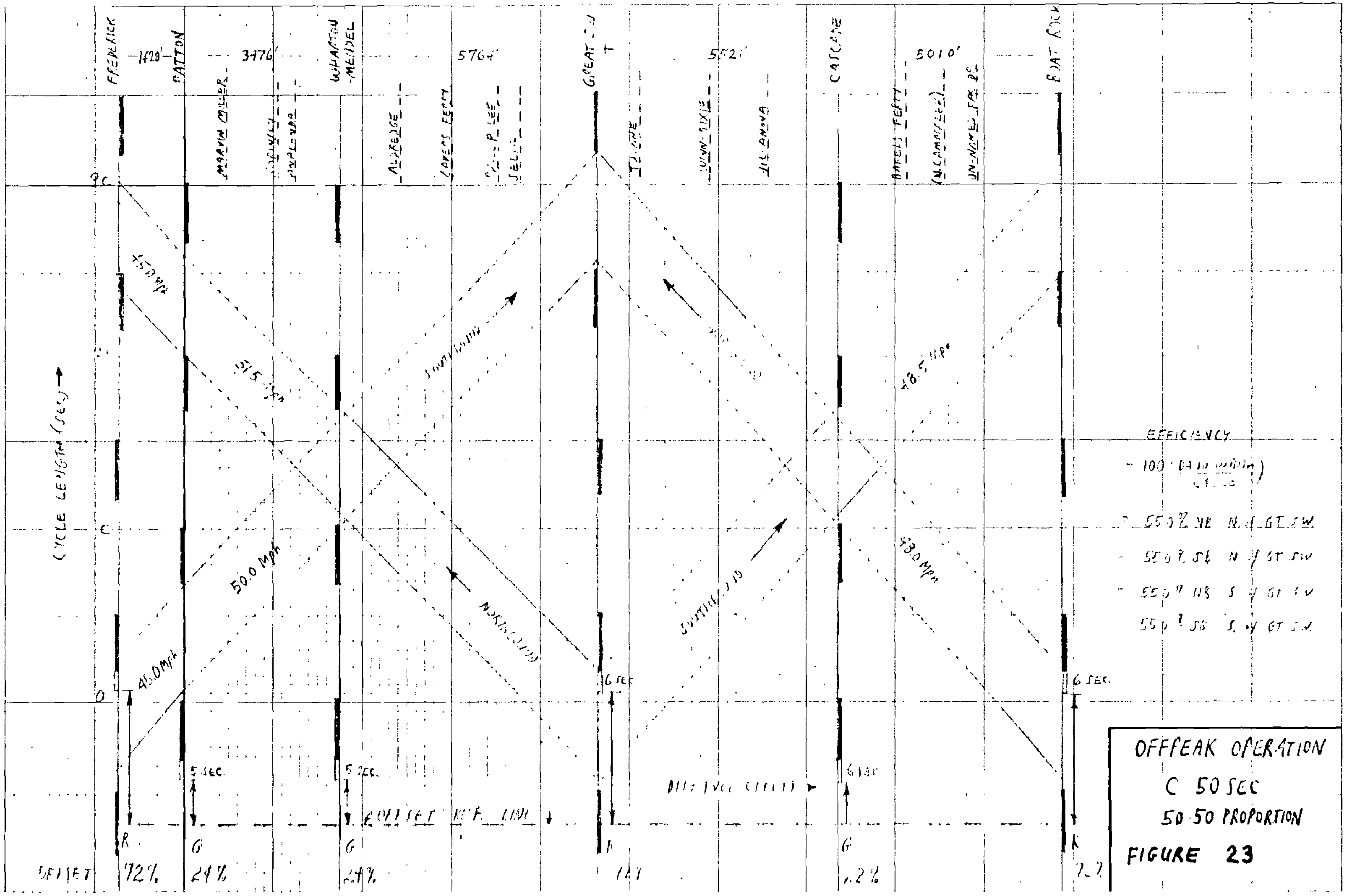
Time-space calculations indicate that cycle lengths of 50, 50, and 60 seconds would provide a level of service "C" or better during the morning, offpeak, and evening periods, respectively, at most locations. Exceptions occur at Patton and Frederick Drives which would need to be double cycled during both the morning and evening peak hours to satisfy a capacity constraint at Patton Drive, as well as the locational constraint at Frederick Drive, discussed below.

Sampling detectors should be located to give an "early warning" of significant impending fluctuations in traffic demand which need to be met by changes in the system cycle length. Four locations need to be sampled as noted in table 16. Such differences occur north and south of the Great Southwest Parkway T intersection during peak traffic periods.

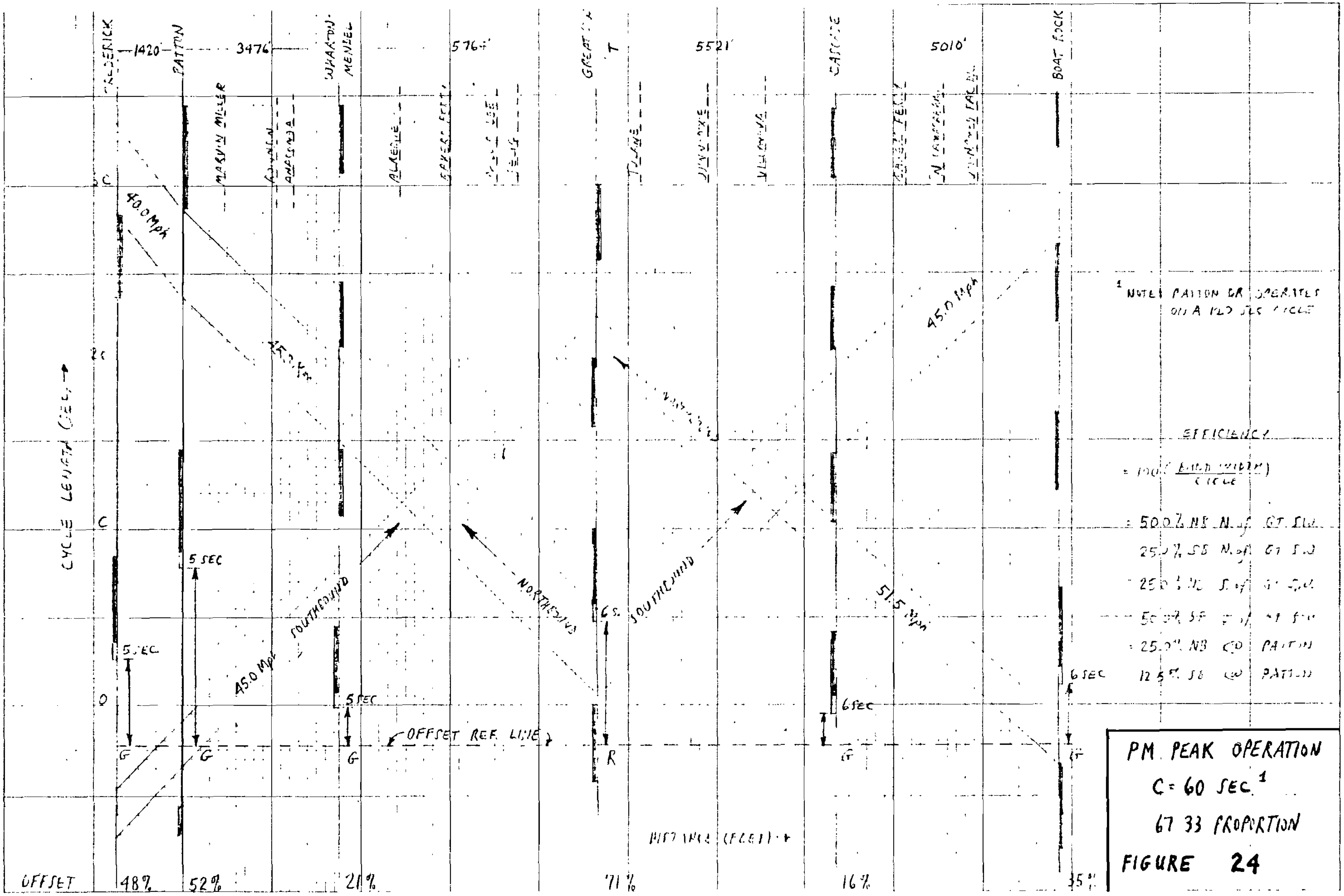
Signal equipment should not be specified at this time, because of rapidly changing technology and the desirability of postponing decisions on hardware until future traffic growth requires its purchase.



AM PEAK OPERATION
C = 50 SEC[1]
70-30 PROPORTION
FIGURE 22



OFFPEAK OPERATION
C 50 SEC
50-50 PROPORTION
FIGURE 23



PM PEAK OPERATION

C = 60 SEC.[1]

67 33 PROPORTION

FIGURE 24

TABLE 16 SAMPLING DETECTOR LOCATIONS

Sampling Direction	Location
Southbound	SB lanes, 300 feet north of Patton Dr.
Northbound	NB lanes, 300 feet south of Boat Rock Rd.
Southbound	SB lanes, just south of Tulane Dr.
Northbound	NB lanes, 100 feet north of Great Southwest Parkway (T).

Recommended Signal Locations

The recommended locations of additional signals in the future progressive system were determined from the time-space diagrams (figures 22 through 24) and by application of the access standards developed in the ANALYSES section.

Tables 17 and 18 exhibit results based solely on the time-space diagrams. Table 17 documents the ability of all intersections,which are candidates for signalization,to meet optimal standards of progressive efficiency as established by the proposed "core" system. This core system includes the existing signalized locations of Patton Drive, Wharton Drive, the Great Southwest Parkway and Cascade Road. Boat Rock Road and Frederick Drive, both presently unsignalized, would provide the anchors to the "core" system. The time-space diagrams were established for optimal bandwidths during peak and offpeak periods along this "core." Locations "A" "B", "C" and "D" (see table 17) represent roadway segments between "core" intersections which will require one or more "spacer" signals to achieve overall progressive movement between Frederick Drive and Boat Rock Road. The number of spacer signals is a function of the segment length and the access standards,which indicate that a range between 1600 feet and 2000 feet is ideal for optimum progression (25). This is generally achieved within the proposed system.

While table 17 provides a handy reference to the relative merits of signalizing one location versus another, table 18 also has a useful function. This latter table establishes that range of distances, as oriented to an adjacent "core" intersection, over which a signal may be installed to meet optimum system efficiency. Table 18 denotes three sets of locational ranges depending upon which time-space diagram was utilized. In actuality, a composite is necessary to arrive at a single acceptable range or value. Thus, an ideal or optimal system COULD be created to take advantage of

TABLE 17

OPTIMIZATION OF FUTURE SIGNAL LOCATIONS

(PROGRESSIVE EFFICIENCY)[1]

.OCATION	AM PEAK				OFF PEAK	PM PEAK			
'CORE" SYSTEM	N. of GT. S.W.		S. of GT. S.W.			N. of GT. S.W.		S of GT. S.W.	
	NB	SB	NB	SB		NB	SB	NB	SB
'rederick Dr.	28%	68%	68%	28%	55%	50%	25%	25%	50%
'atton Dr.									
"A"...									
!harton Dr.									
"B"...									
;T. S.W. Pkwy									
"C"...									
;ascade Rd.									
"D"...									
.oat Rock Blvd.									
'A" Options 'Select 1 .ocation)									
!arvin Miller Dr.	23	68	–	–	45	50	0	–	–
:obinson Dr.	28	68	–	–	55	50	16	–	–
.naconda (Priv. Dr.)	25	68	–	–	45	50	0	–	–
'ptimal	28	68	–	–	55	50	25	–	–
B" Options Select 2 .ocations)									
.ldredge Blvd.	8	68	–	–	46	50	0	–	–
.aker's Ferry Rd.	28	68	–	–	53	50	20	–	–
'hillip Lee Dr.	19	68	–	–	55	50	18	–	–
elig Dr.	28	68	–	–	55	50	25	–	–
'ptimal - 1st Signal	28	68	–	–	55	50	25	–	–
'ptimal - 2nd Signal	28	68	–	–	55	50	25	–	–

(1) By percent efficiency of band widths (2) Presently unsignalized

TABLE 17 (Continued)

.OCATION	AM PEAK				OFF PEAK	PM PEAK			
CORE" SYSTEM	N of GT. S.W.		S. of GT. S.W.			N. of GT. S.W.		S. of GT. S.W	
	NB	SB	NB	SB		NB	SB	NB	SB
'C" Options 'Select 2 .ocations)									
'ulane Dr.	–	–	68	5	40	–	–	0	50
'inn-Dixie Priv. Dr.)	–	–	68	5	45	–	–	0	50
'illanova Dr.	–	–	68	28	55	–	–	25	50
ptimal - 1st Signal	–	–	68	28	55	–	–	25	50
ptimal - 2nd Signal	–	–	68	28	55	–	–	25	50
'D" Options 'Select 2 .ocations)									
aker's Ferry Rd.	–	–	68	28	55	–	–	0	50
'N. Campcreek Pkwy.)	–	–	68	14	43	–	–	25	50
'n-Named Fact. (Priv. Dr.)	–	–	68	26	55	–	–	12	50
ptimal - 1st Signal	–	–	68	28	55	–	–	25	50
ptimal - 2nd Signal	–	–	68	28	55	–	–	25	50

TABLE 18

OPTIMIZATION OF FUTURE SIGNAL LOCATIONS

(POTENTIAL LOCATION RANGE)

	AM PEAK	OFF PEAK	PM PEAK	OPTIMAL COMPOSITE
"A" Option				
S. of Patton Dr.	1900'-2500'	1700'-1900'	1500'-1950'	1900'
"B" Options				
S. of Wharton Dr.	1900'-2500'	1900'-2100'	1900'-2450'	1900'-2100'
N. of GT. S.W. Pkwy	1350'-2000'	1750'-2000'	1400'-1850'	1750'-1850'
"C" Options				
S. of GT. S.W. Pkwy	1450'-2100'	1750'-1900'	1500'-2000'	1750'-1800'
N. of Cascade Rd.	1500'-2100'	1500'-1800'	1350'-1900'	1800'
"D" Options				
S. of Cascade Rd.	1400'-2100'	1600'-1750'	2300'-2900'	1600'-1750'
N. of Boat Rock Rd.	1100'-1800'	1550'-1700'		

the best possible progression if no locational constraints occurred. Such an option is shown for each required signal location in table 17 together with its attainable efficiency. For example, the "optimal" location noted under the "A" options can be achieved if located within the composite range noted in table 18.

From a practical viewpoint, partial development presently in place along Fulton Industrial Boulevard makes such an optimal system unlikely to be attained. Nonetheless, where possible, it is considered preferable to orient future development around the proposed signal system.

As an example, consider the three existing intersection locations of segment "A". A fourth option exists in an optimal location, in this case 1900 feet south of Patton Drive. Two of the locations, Marvin Miller Drive and the Anaconda driveway are first eliminated by their failure to meet minimum spacing requirements from "core" intersections. It remains to be determined whether the choice of Robinson Drive would materially affect system efficiency versus the optimal solution. Examination of table 17 indicates no difference during the morning peak period and the offpeak intervals. In the evening, some reduction would be experienced for southbound traffic (from 25 to 16 percent); however, this does not warrant scrapping Robinson Drive in favor of a completely new location.

In segment "B" a pair of spacer signals will ultimately be required to provide proper progression. The south signal is satisfied directly by Selig Drive. Bakers Ferry Road is seen to be superior to Aldredge Boulevard in all efficiency categories. Although some reduction in offpeak bandwidth results, this intersection is preferred over the optimal location. Terrain considerations would impose restrictions on relocation of Bakers Ferry Road. Existing development also restricts any effective redesign of this access point.

Segment "C" also has a potential need for two additional spacer signals. At the south end Villanova Drive reasonably approximates optimal conditions. To the north neither Tulane Drive nor the Winn-Dixie terminal driveway represents a good location for a future signal, without seriously constricting peak hour flow in the lighter direction. A significant decline in offpeak system efficiency would result from selection of either of these access points. Adjacent land uses are predominantly vacant to the north where a new intersection should be provided (ie. 1750' - 1800' south of the Great Southwest Parkway). A four way intersection is suggested to open development opportunities either side of the Boulevard.

Segment "D" options include the existing intersections with the south leg of Bakers Ferry Road and a private driveway. North Camp Creek Parkway is proposed to intersect the Boulevard about 830 feet south of Bakers Ferry Road. Review of tables 17 and 18 indicates that two signals could be located at Bakers Ferry Road and the private driveway which would satisfy morning and off-peak requirements. The evening peak could be best satisfied by a single signalized intersection at N. Camp Creek Parkway. This apparent dilemma can be solved by considering that selection of the aforementioned two signals would still satisfy evening peak period requirements in the predominant direction of flow. From a practical viewpoint, the Parkway must ultimately be signalized. Relocation of the Parkway intersection to a range of 100 to 250 feet south of Bakers Ferry Road establishes that pair of signals (the other being at the private drive) which would function most efficiently in terms of the overall system. In this case, Bakers Ferry Road (S. leg) would either need to be channeled via a frontage road to Cascade Road/Great Southwest Parkway or dead-ended. It is recommended that the frontage road concept be utilized west of the Boulevard and dead-ending be used to the east. This concept would be compatible with existing land use which is predominantly residential eastward along Bakers Ferry Road.

Figure 25 illustrates proposed signalization for the future along the length of the study area. Geometric alterations and the resultant traffic diversions are discussed under future improvements of specific intersections or areas. Broad geometric recommendations appear in figure 26 for the entire study section.

Future Intersection Improvements

Substantial geometric imporvements will be ultimately necessary, in some cases, to assist the progressive system in attaining high levels of efficiency. In other locations, to the north of and including the I-20 interchange, capacity and storage problems will need to be resolved by lengthening of left turn bays and widening of approaches. Changes in signalization control are discussed at the I-20 ramps.

Gordon Road will continue to operate on an isolated basis as far as can be foreseen. The northbound Boulevard to westbound Gordon Road left turn bay will need to be lengthened to handle projected traffic. By 1980 it is estimated that 640 vehicles will be using the turn slot during the evening peak period. At 10 percent trucks and with a cycle length of 60 seconds, 425 feet of full width turn bay will be needed.

Interchange Drive should be provided with a left turn slot, as noted in figure 27, to handle turns presently made from the high speed lane. By 1980 about 150 vehicles are forecast to be making this maneuver.

The I-20 interchange is expected to function on a coordinated basis, though isolated from Gordon Road and Frederick Drive. A new control strategy may be necessary if the pair of 1826N(M2) controllers and the CR-16 coordinating unit do not prove efficient. This approach would consist

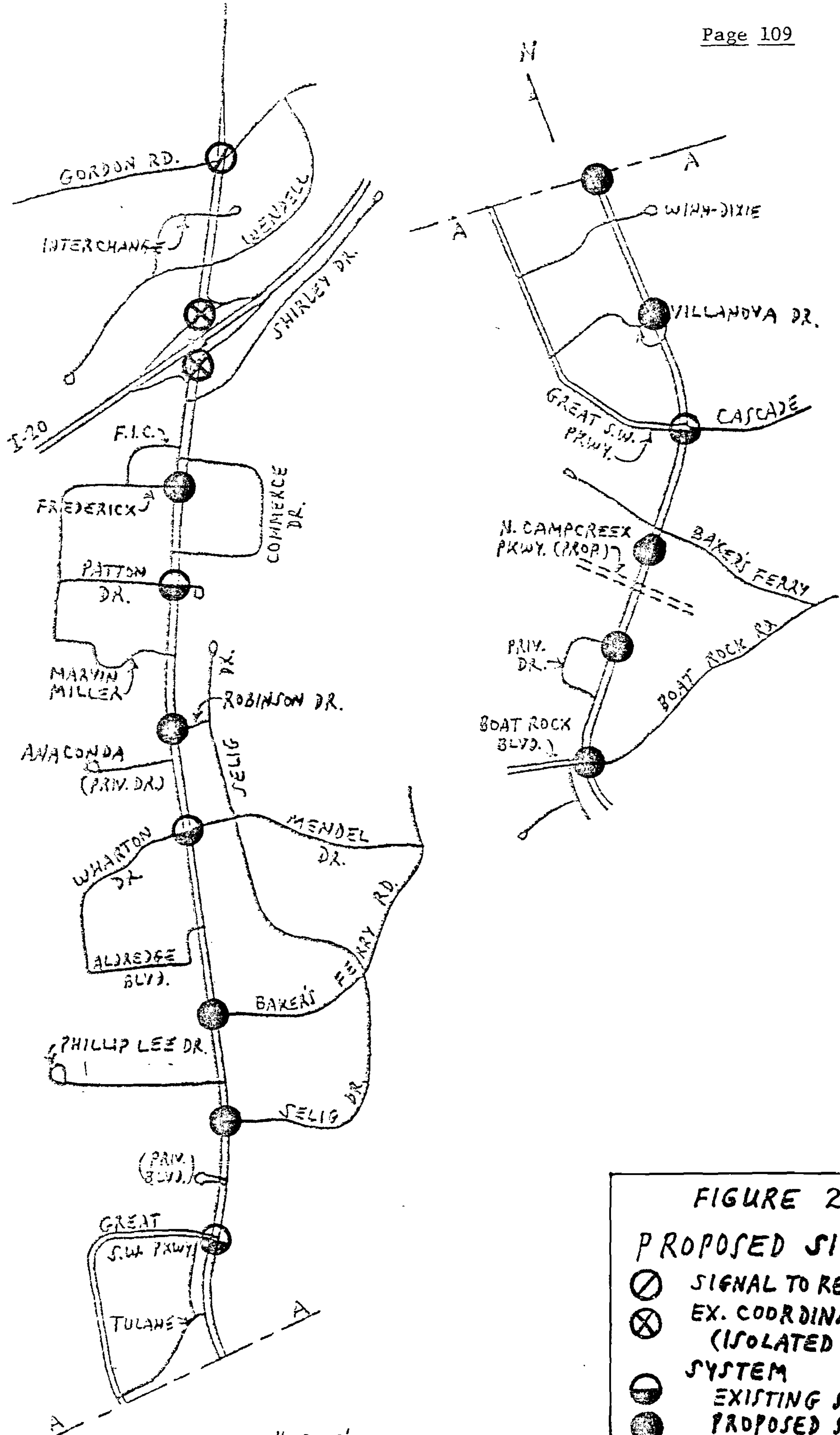
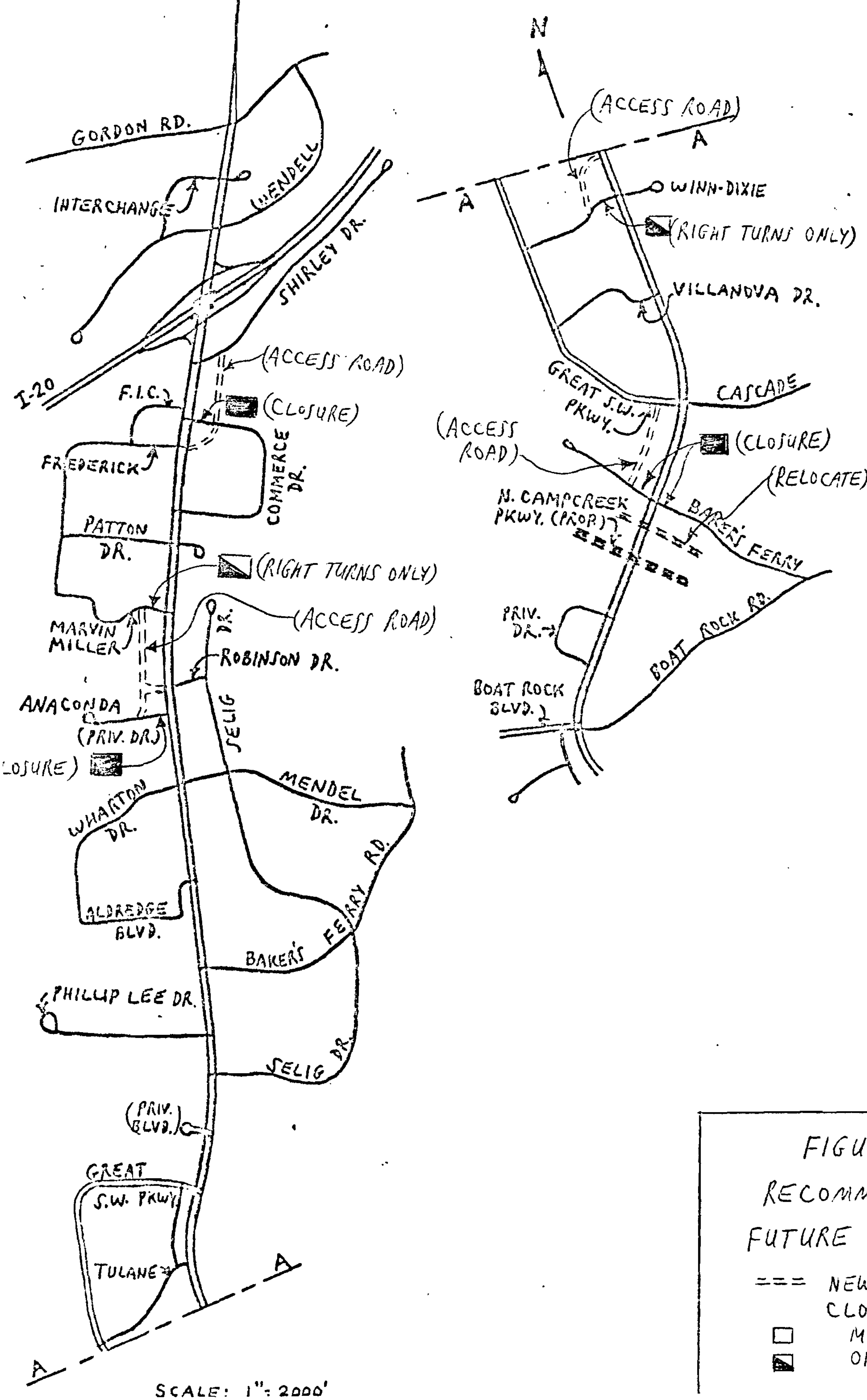


FIGURE 25

PROPOSED SIGNALIZATION

- ⊘ SIGNAL TO REMAIN ISOLATED
- ⊗ EX. COORDINATED DIAMOND (ISOLATED FROM SYSTEM)
- SYSTEM
  - EXISTING SIGNALS
  - PROPOSED SIGNALS



SCALE: 1" = 2000'

FIGURE 26
RECOMMENDED
FUTURE GEOMETRICS

=== NEW ACCESS ROADS
CLOSURES
☐ MANDATORY
▨ OPTIONAL

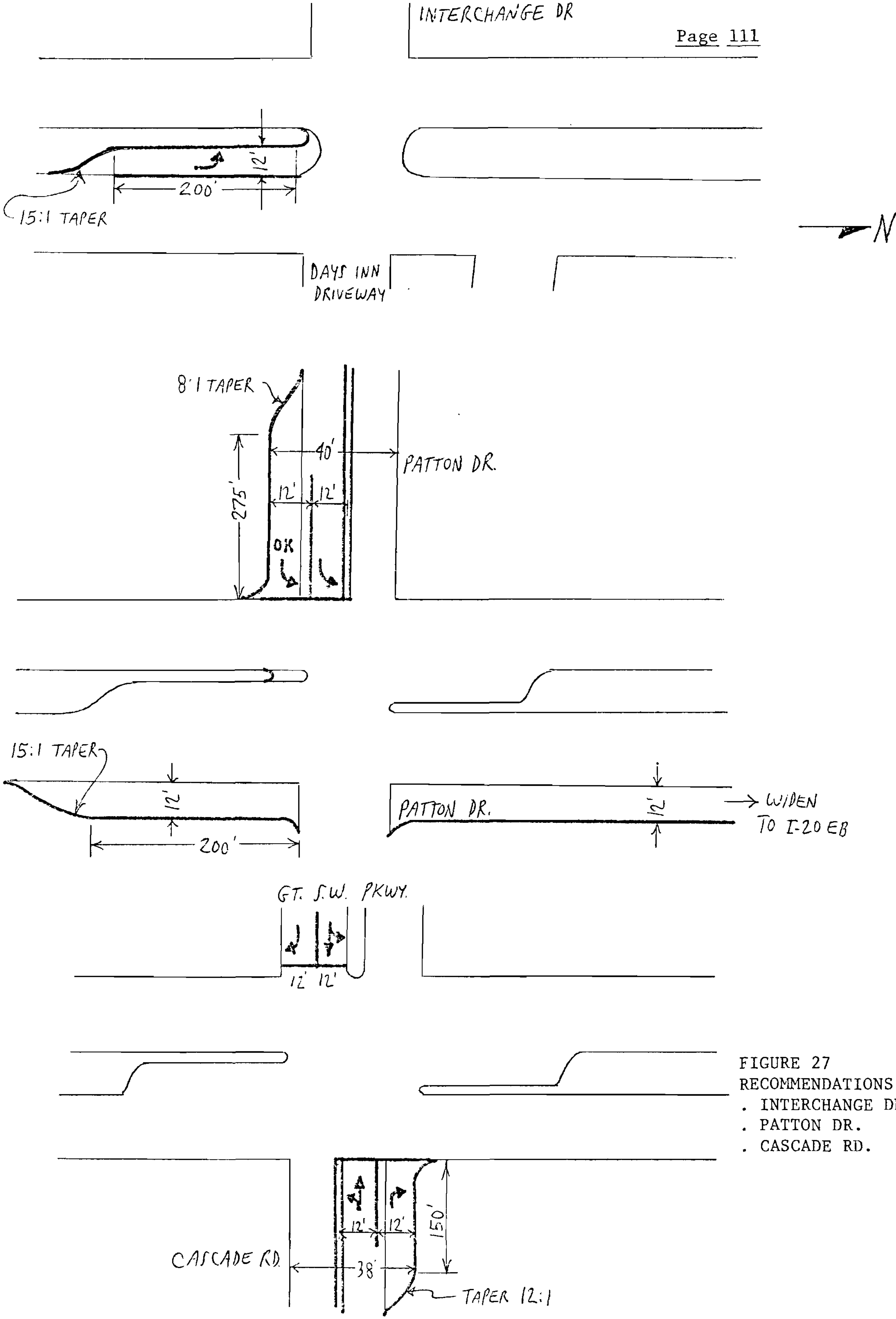


FIGURE 27
RECOMMENDATIONS
. INTERCHANGE DR.
. PATTON DR.
. CASCADE RD.

of using a two phase controller and a pair of minor movement controllers whose function would be to regulate the ramps. The system would be a four phase operation, plus a pair of overlaps, as developed by the Texas Transportation Institute. The detailed phasing sequence appears in figure 14 in the ANALYSES section. The pair of overlaps and double clearance intervals tends to effectively flush interior movements and increases usable green time.

As volumes increase the installation of a queue detector for the westbound off ramp will become essential. The detector should be strategically located for proper operation.

Heavier demands will also require the widening of the northbound Boulevard from the eastbound I-20 ramps as far south as Patton Drive. At I-20 the existing 10 foot shoulder, presently used for right turns, should be widened to a 12 foot turn lane. Double right turns should be permitted by construction of a second 12 foot lane commencing just north of Shirley Drive (see figure 28).

The area from Shirley Drive to Frederick Drive will require numerous geometric improvements as well as signalization to reduce cross movement accident hazards associated with these streets, as well as the north leg of Commerce Drive. In 1974, twenty-seven such collisions occurred. These accidents primarily result when left turn traffic from minor streets attempts to find gaps in very heavy peak hour traffic. Numerous serious injuries have resulted as noted in the accident analysis. Geometric improvements would include; closure of the median at Shirley and Commerce Drives, closure of private drives on the Boulevards' east side between Frederick Drive and the eastbound I-20 ramps, relocation of Commerce Drive to an alignment opposite Frederick Drive, and construction of a frontage road between Shirley and Commerce Drives. This frontage road would provide additional

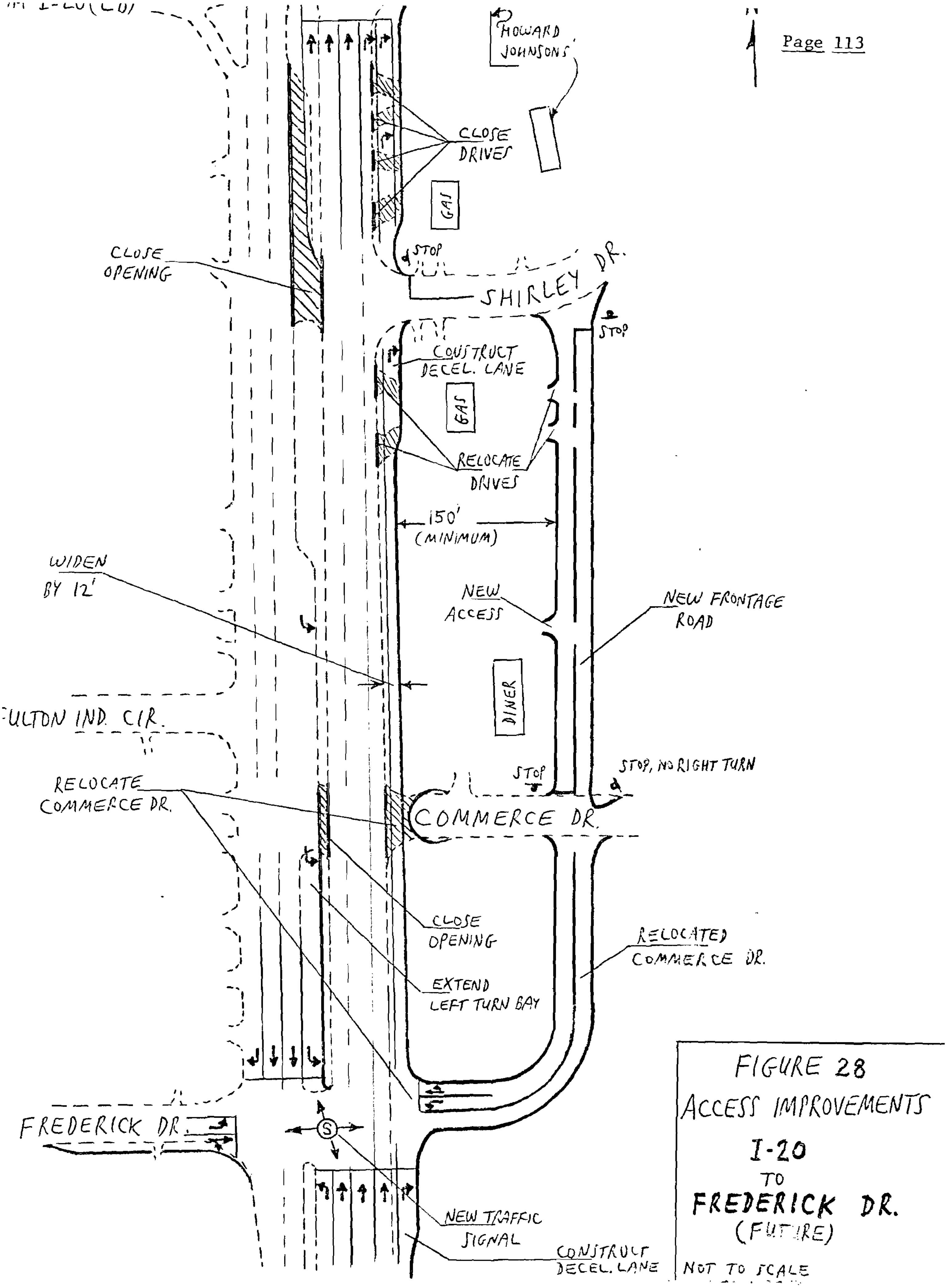


FIGURE 28
ACCESS IMPROVEMENTS
I-20
TO
FREDERICK DR.
(FUTURE)
NOT TO SCALE

access to some properties whose drives facing the Boulevard would be closed. In no case would any property be denied access altogether. An extended left turn bay (SB) will be adequate to handle left turns to relocated Commerce Drive. Deceleration lanes should be provided for right turns onto Shirley and Commerce Drives.

Turn restrictions would be imposed at Shirley Drive (right turn only) and at the intersection of Commerce Drive and the proposed frontage road. Commerce Drive (westbound) traffic would be prohibited from turning onto the frontage road. This would eliminate most potential for conflicts and delays at the intersection of Shirley Drive and the frontage road.

Traffic signals should be warranted at least by 1980 based on a straight line projection of future traffic growth and consideration of the benefits of accident reduction. The only new signals between Patton Drive and I-20 should be located at Frederick Drive. A split-phase operation is proposed with a protected movement given to southbound Boulevard left turn traffic by an exclusive display. Sight distance restrictions prevent the use of a permissive display. It is necessary that the northbound approach be widened by 12 feet to handle capacity demands (this is the case from the I-20 (EB) ramps to Patton Drive).

This project should be given an early priority relative to other future geometric recommendations because of the associated accident reduction benefits anticipated and the fact that this section represents the major Boulevard bottleneck during the evening rush period.

Proposed improvements appear in figure 28, while figure 29 documents traffic diversions expected if this circulation pattern were to be enacted under present volumes.

Patton Drive should be widened on its eastbound approach to accommodate increased traffic demands and to allow a better fit to overall signal system

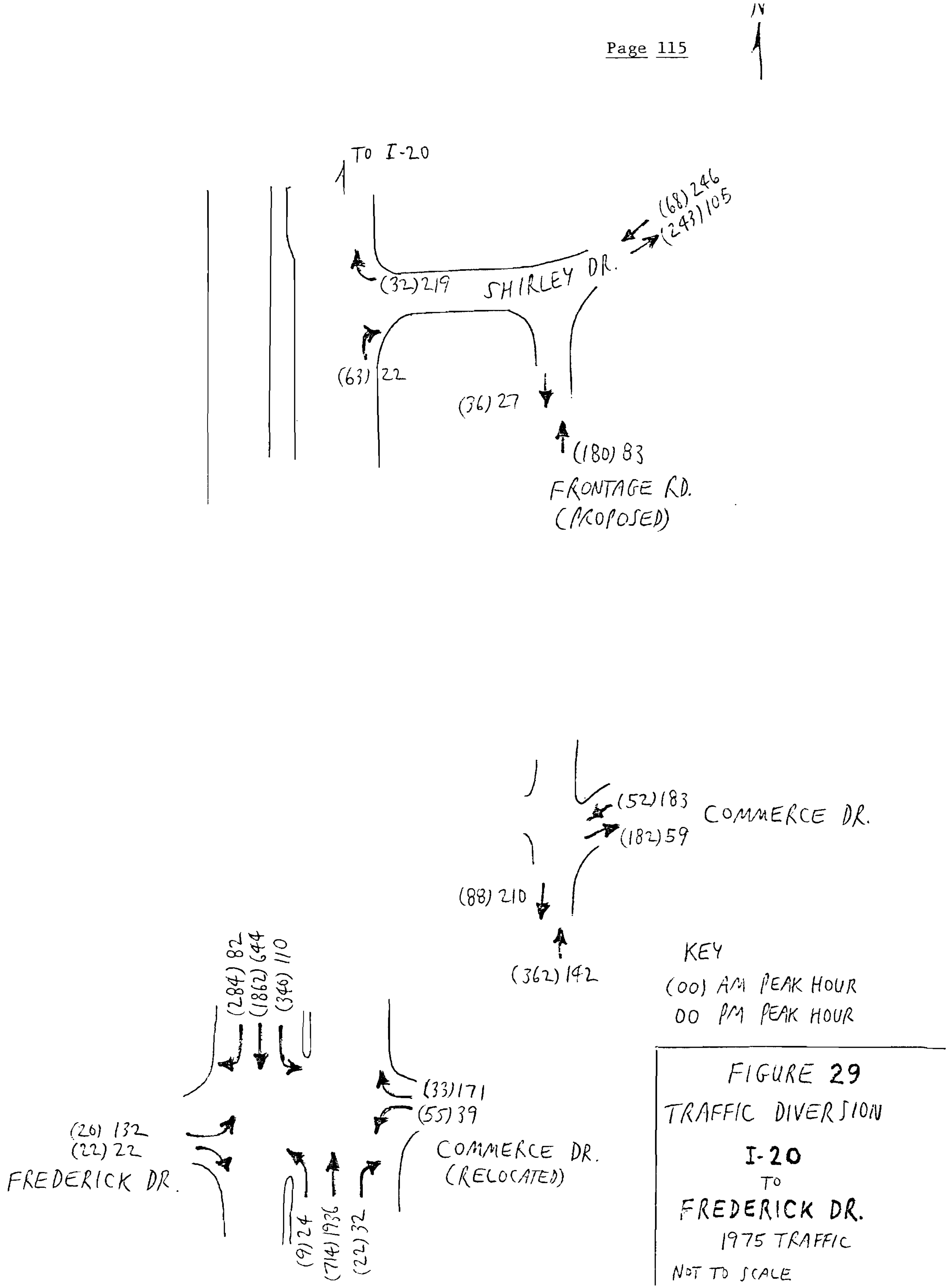


FIGURE 29

TRAFFIC DIVERSION

I-20

TO

FREDERICK DR.

1975 TRAFFIC

NOT TO SCALE

progressive efficiency. By 1980 left turns from Patton Drive are estimated to reach 200 vehicles during the evening peak hour. Based on 20 percent trucks and a 120 second signal cycle length, two approach lanes of full 12 foot width should be provided for a distance of 275 feet behind the stop bar. The widening of the Boulevard's east side should be necessary only to the limits shown in figure 27, based on anticipated 1980 traffic. This figure also illustrates other future recommendations.

Robinson Drive is proposed to be the location of a new traffic signal. The presense of a fire station driveway, about 150 feet east of the Boulevard, means that preemption control may need to be provided at this intersection. Marvin Miller Drive and the private road serving Anaconda are proposed to be aligned directly opposite Robinson Drive at a single access point. Although only four accidents were recorded at Marvin Miller Drive in 1974, its location presents a sight distance problem to the south for exiting left turn traffic during the evening peak hour. Anaconda traffic causes accident hazards at Wharton-Mendel Drives following factory shift changes. Although the Anaconda shift changes do not occur simultaneously with the evening street peak hour, problems result due to existing access conditions. Most Anaconda traffic desires to travel northbound, as observed during field counts, but are denied access via a median opening. Thus, they travel south as far as Wharton Drive and make "U" turns whenever they can find a gap both with and against the green. Anticipated traffic diversion and recommended geometrics appear in figure 30.

Signal control should consist of a three phase operation with phase C providing an exclusive left turn display simultaneously to northbound and southbound Boulevard traffic. High speed traffic will not permit the

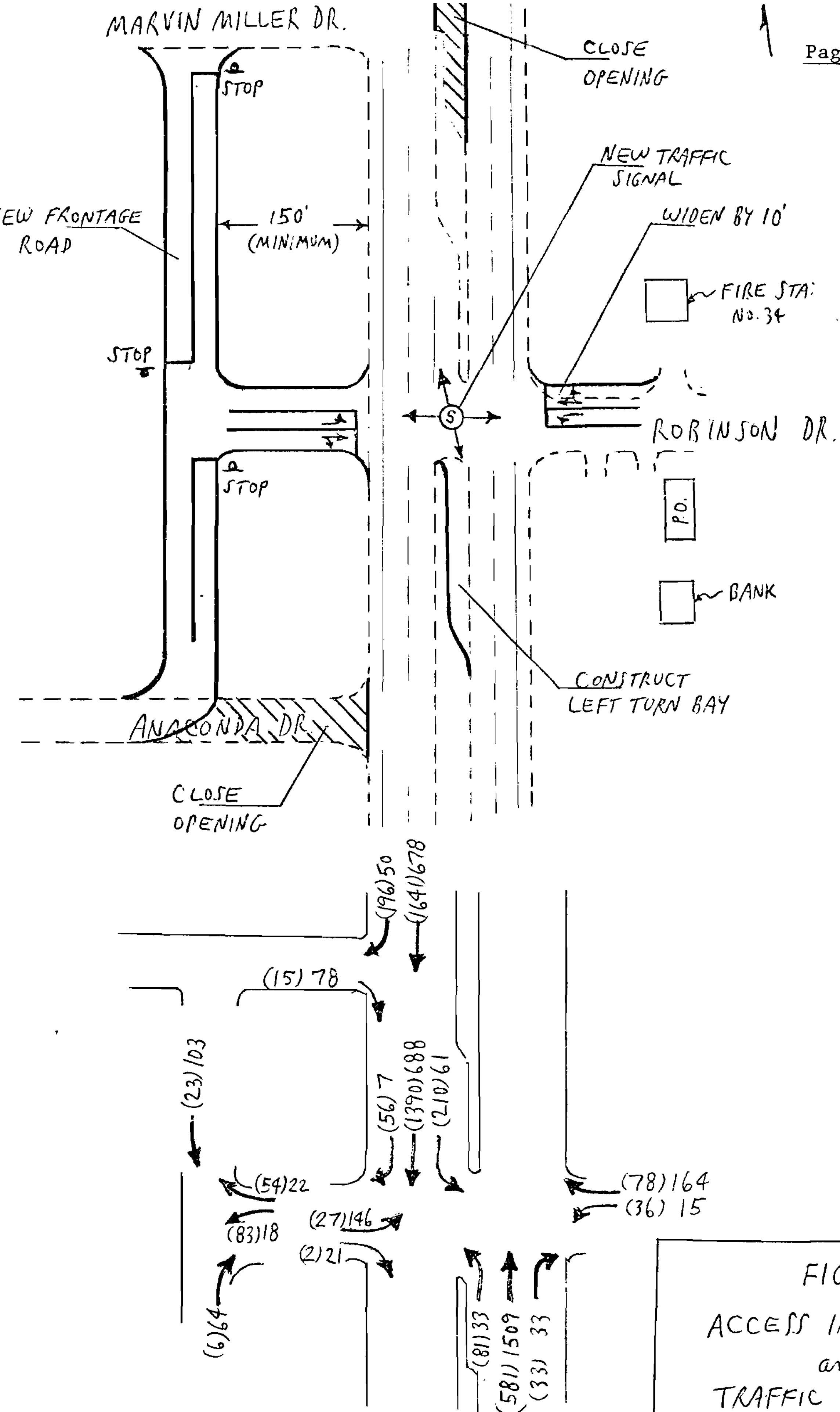


FIGURE 30

ACCESS IMPROVEMENTS and TRAFFIC DIVERSION

ROBINSON DRIVE AREA

(FUTURE)

NOT TO SCALE

use of a permissive display.

The proposed frontage road approach to the Boulevard would consist of a pair of 12 foot lanes designated as in figure 30. Robinson Drive should be widened from 30 to 40 feet to allow a pair of 12 foot approach lanes. The median would be closed at Marvin Miller Drive, although right turns would continue to be served. Anaconda's private road would be closed at the Boulevard and joined to the frontage road. Stop sign control would be provided along the frontage road as appears in figure 30.

The north leg of Bakers Ferry Road is recommended as one of the spacer signal locations between Wharton Drive and the Great Southwest Parkway T intersection. This intersection as well as Selig Drive, the second proposed spacer signal location, will not require any foreseen geometric changes. Signal phasing will be dependent on future traffic conditions but each location is likely, based on 1980 projections, to require a two-phase, semi-actuated controller.

A new signalized location will need to be established between 1750 and 1800 feet south of the Great Southwest Parkway T intersection. This location is necessary to provide proper progression of the proposed signal system. The intersection would also satisfy access spacing requirements. Since no development currently exists on either side of this proposed location, about 600 feet south of the Utoy Creek bridge, signal control and intersection geometry will be dependent on future demands. A right-of-way should be reserved extending from the access road opposite Winn Dixie northeasterly to the Boulevard (see figure 26). Once this new intersection is signalized it is likely that left turns would be prohibited from the eastbound approach of the access point opposite Winn Dixie, instead being diverted to the new intersection.

Villanova Drive will be the location of the second spacer signal between the Great Southwest Parkway T intersection and Cascade Road. No changes in geometry are likely, to meet 1980 projected traffic demands. Signalization would consist of a two-phase, semi-actuated controller.

Cascade Road will tend to constrict the efficiency of a progressive signal system if its approach is not widened. Based on morning peak hour conditions of 250 right turn vehicles (est. 1980), a 50 second signal cycle, and 10 percent trucks, a widening extending 150 feet behind the stop bar is recommended. Figure 27 shows the suggested improvements.

The proposed location of North Camp Creek Parkway should be shifted northward to a site within 100 to 250 feet south of Bakers Ferry Road (south leg). This location provides a progressive pattern which would increase overall system efficiency during offpeak periods by over 25 percent (see table 17). Signalization and geometrics should eventually be established at this proposed intersection to effectively handle traffic demands while providing a high degree of system efficiency. Possible changes in traffic circulation patterns may be affected by the construction of this parkway, which would require future analysis of the Boulevard traffic situation.

A private factory driveway would be the other spacer signal required between Cascade Road and Boat Rock Road. This intersection will likely require a two phase semi-actuated controller. Volume data was not taken at this site since the factory opened after data collection had been completed for analysis.

Boat Rock Road should be upgraded from a pair of intersection control beacons to a two-phase, semi-acutated operation. Heavy left turn traffic anticipated to exit from the eastbound approach during the evening peak hour should be provided for by pavement marking and signing additions. A double left turn should be permitted if updated traffic counts indicate that the opposing Boat Rock Road approach volume remains relatively light.

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